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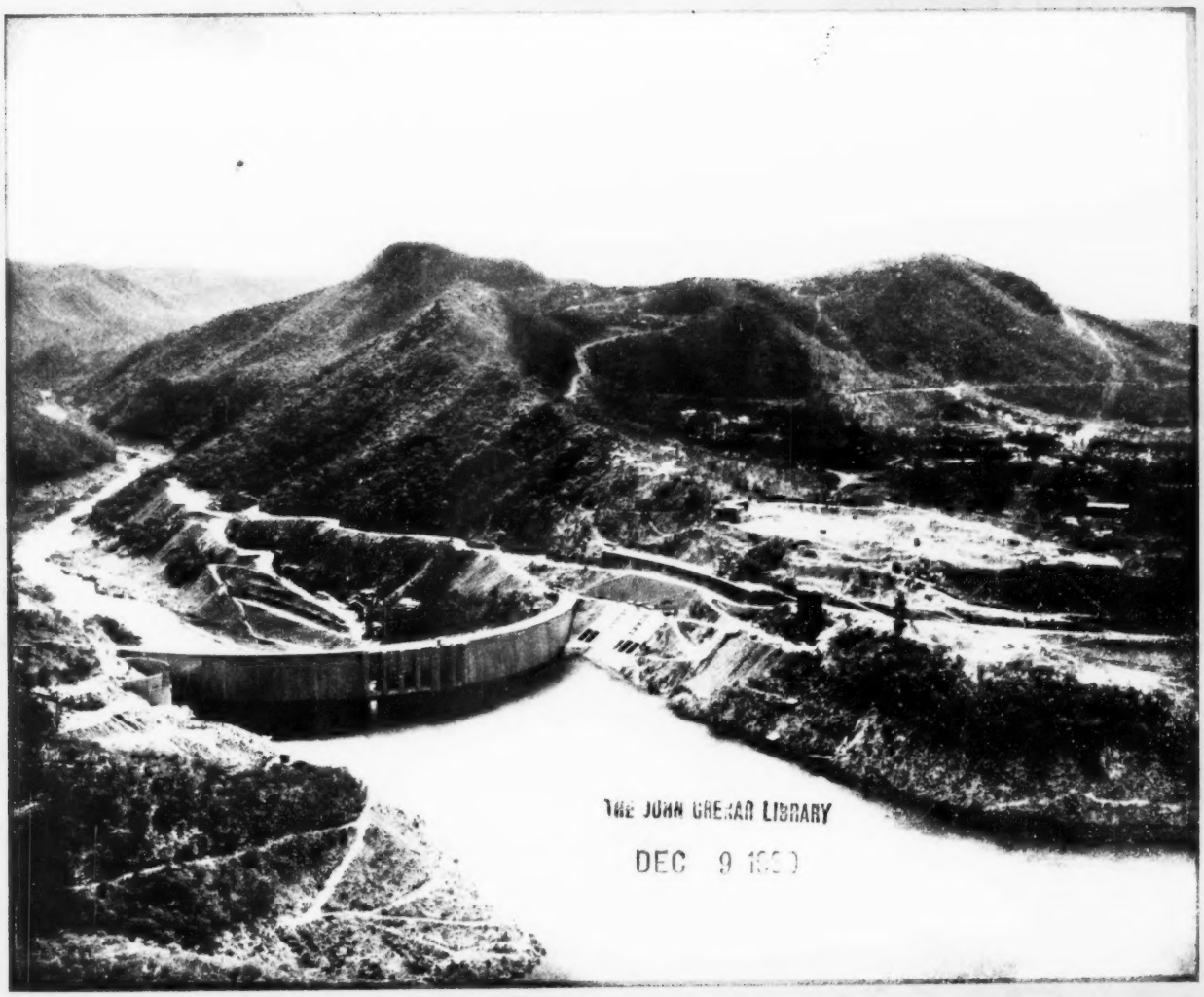
INDIAN JOURNAL OF POWER & RIVER VALLEY DEVELOPMENT

VOL. X NO. 7 JULY, 1960 RUPEES TWO



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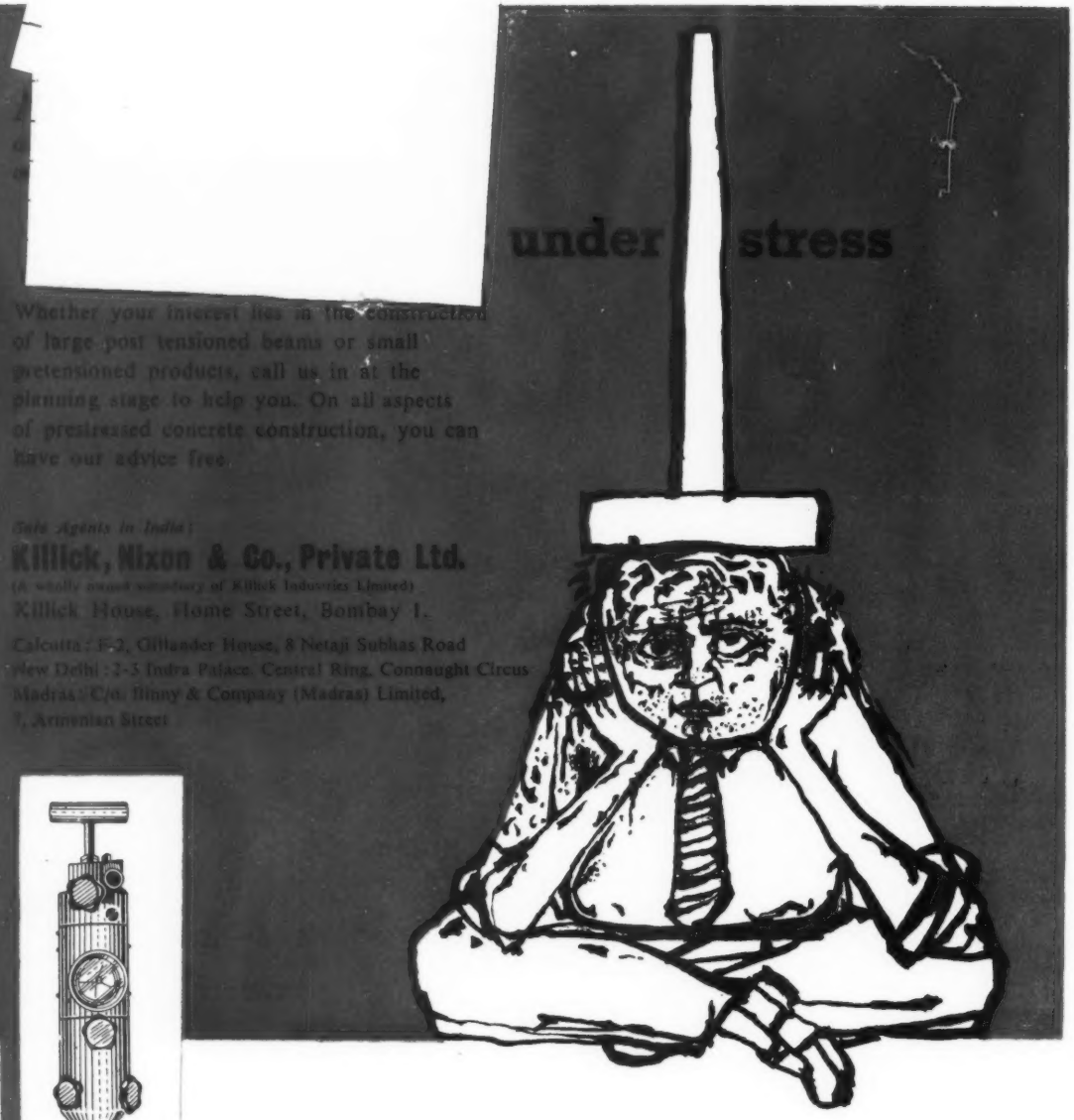

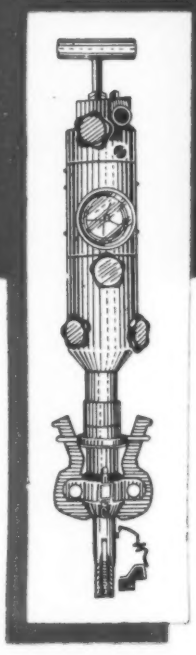

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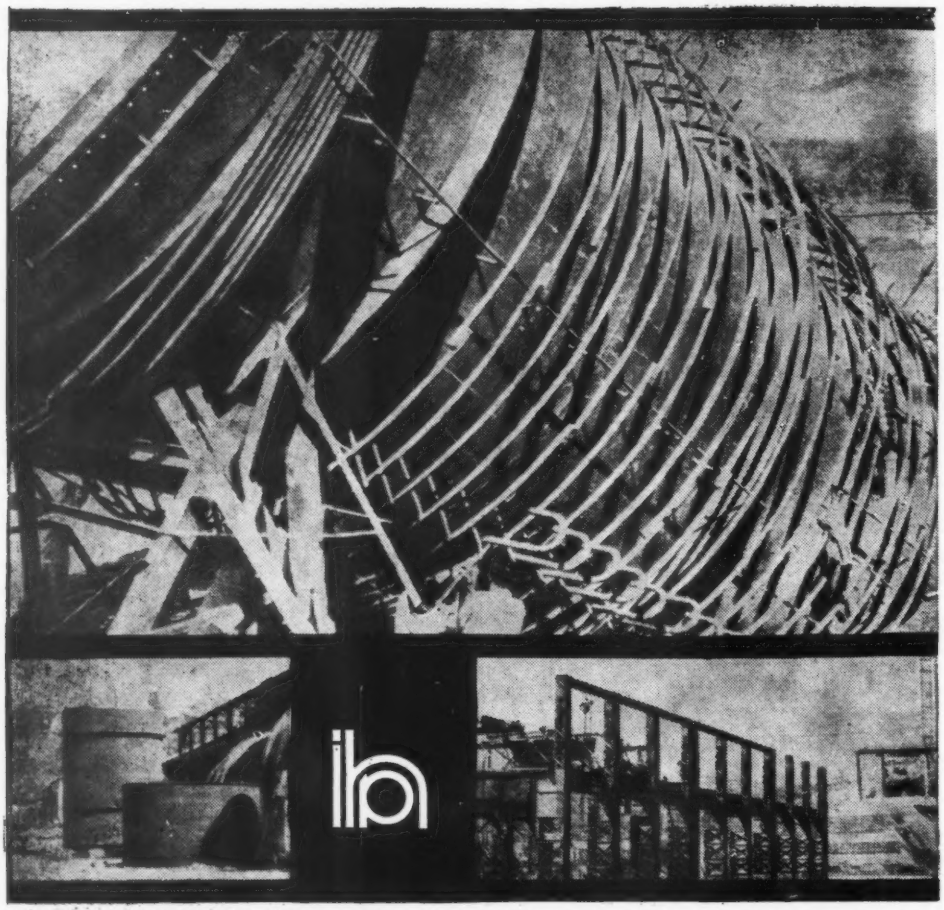

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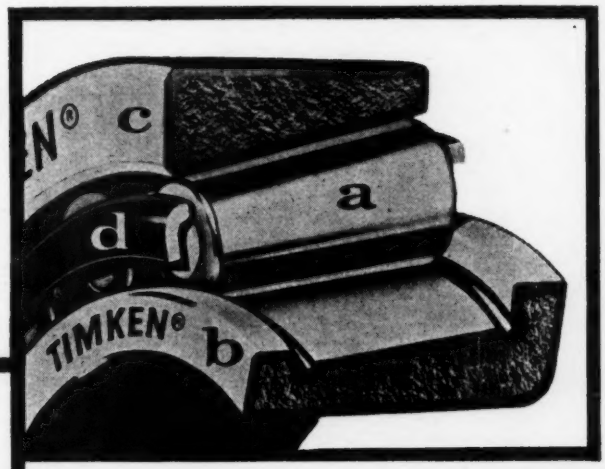
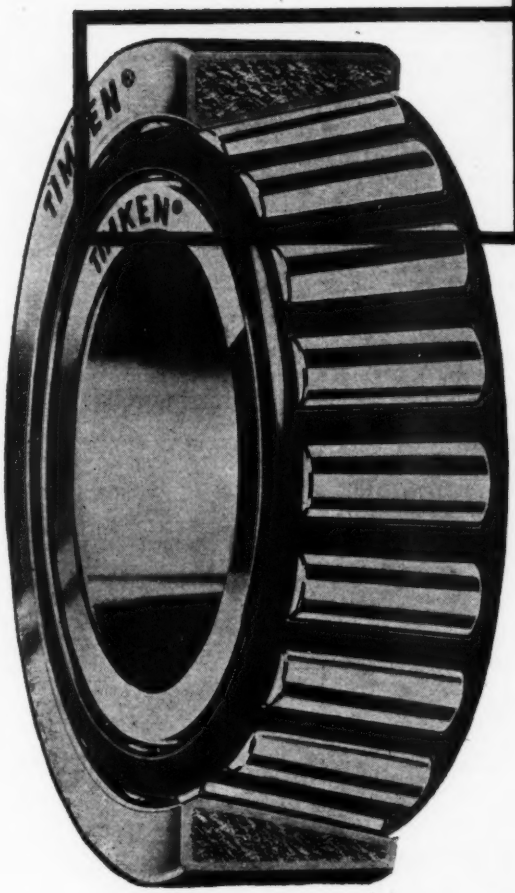
WHAT IS A TIMKEN TAPERED ROLLER BEARING?





It's a roller bearing which is tapered in design so that it can take radial and thrust loads or any combination. At the right is a cutaway view of a portion of a Timken bearing showing one of the tapered rollers (a), the inner (b), and outer (c), races and cage (d). The tapered design of Timken bearings enables them to roll the load and distribute the load along a full line of contact between rollers and races. That's why a Timken tapered roller bearing often does the work of two ordinary bearings—eliminates the need for thrust devices. And Timken bearings are geometrically designed and precision manufactured to roll true.

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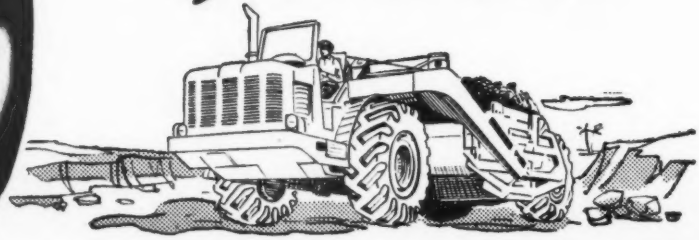

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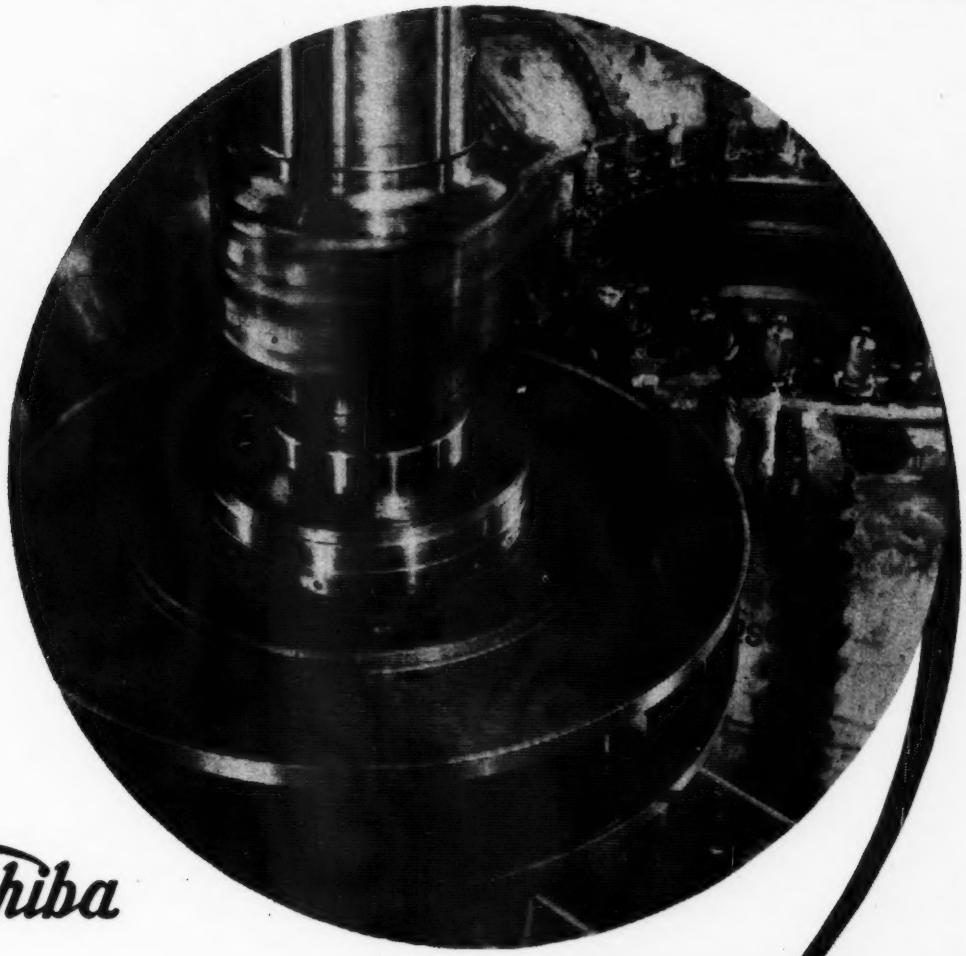

Toshiba

The generator on the right is the second largest of its type in the world. It was built by Toshiba for installation at Japan's new development project, the Okutadami Power Station. It generates 133,000 kVA and was designed big to do a big job. For information about this or any other type of equipment necessary for generating or transmitting electric power, be sure to consult Toshiba.

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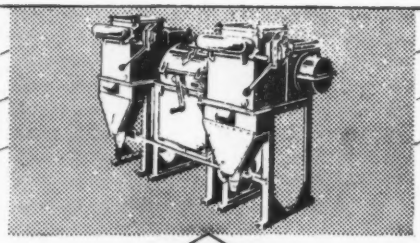

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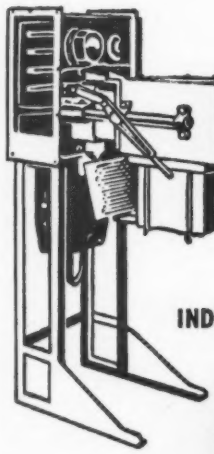
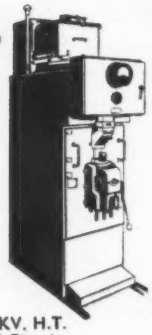
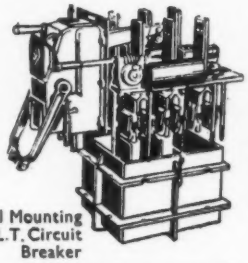


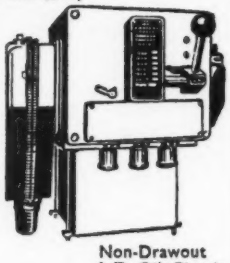
Horizontal L.T. Drawout Circuit Breaker



11 KV. H.T. Oil Circuit Breaker

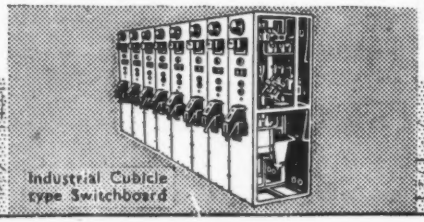


Panel Mounting L.T. Circuit Breaker



Non-Drawout L.T. Oil Circuit Breaker

IA-50



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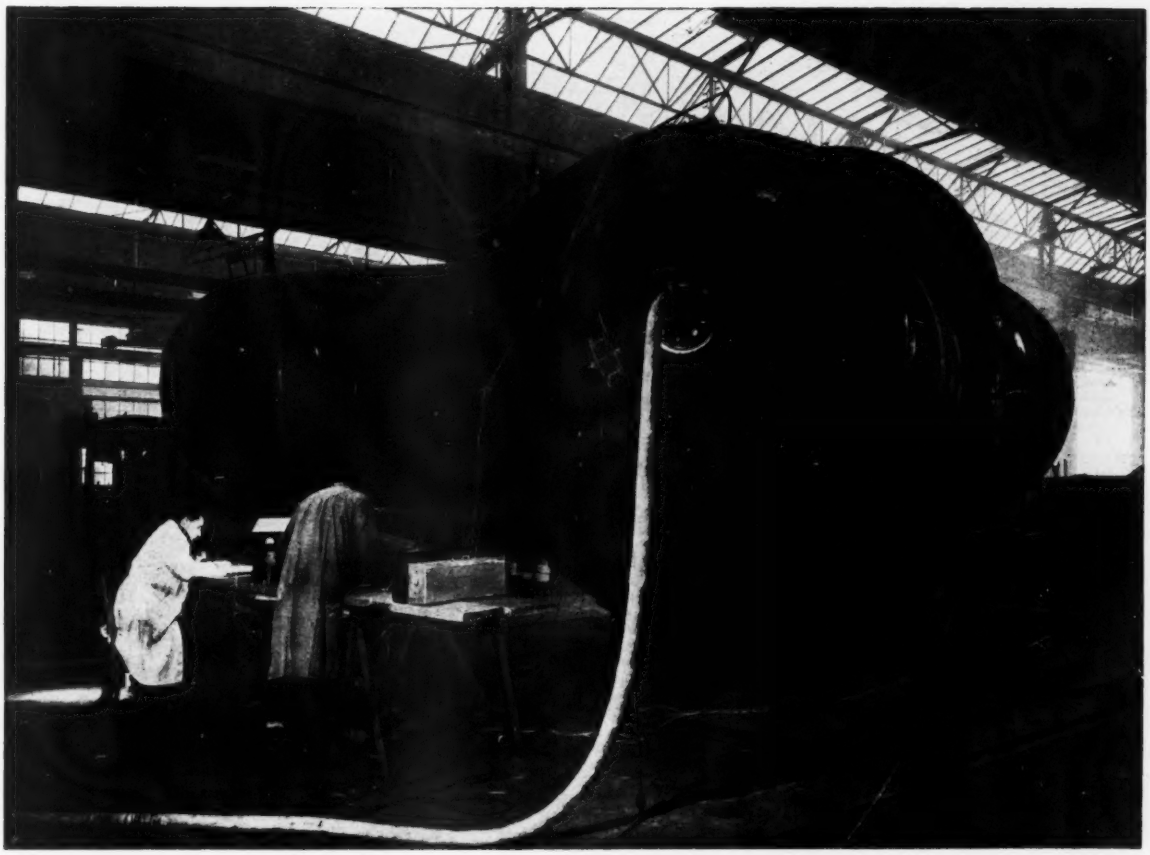
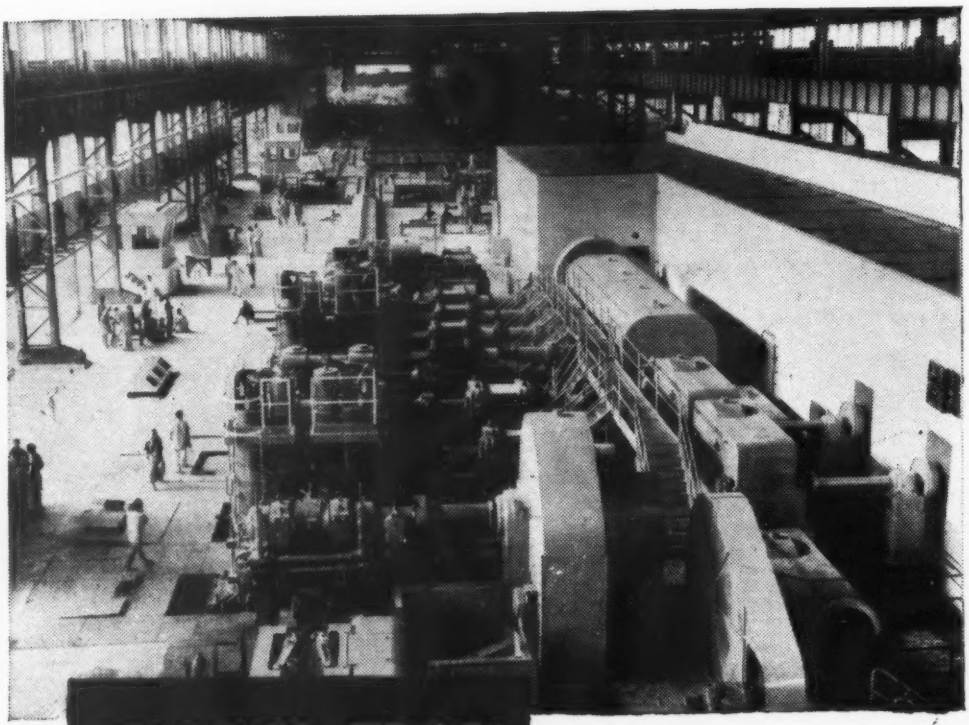


Pressure test and Electric Strain Measuring carried out on the manifold of the KOYNA POWER STATION (Maharashtra State). Test pressure 1420 psi. Diameters 8'6"/2×6'. Thickness 1" 27/32 and 1" 37/64. Weight 40·5 tons.

GIOVANOLA Monthey - Switzerland



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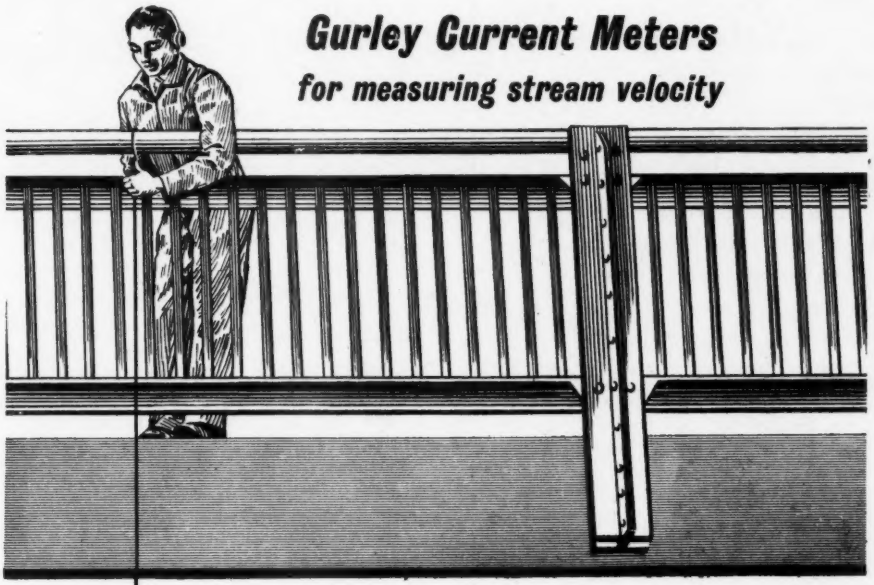
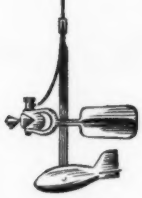


Gurley Current Meters
for measuring stream velocity



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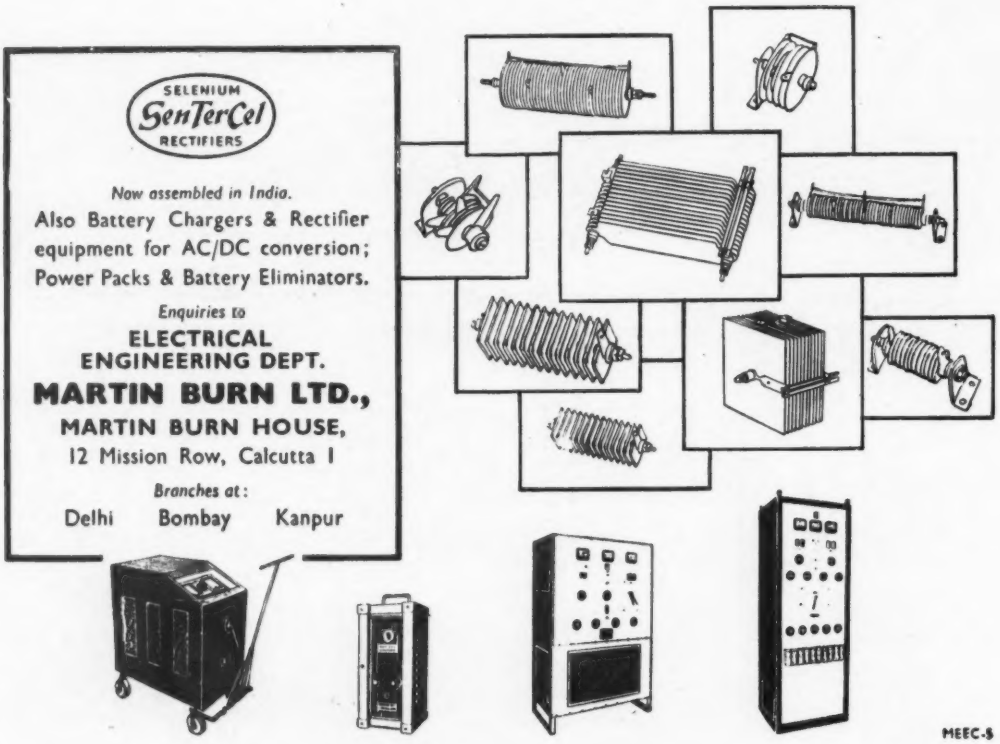

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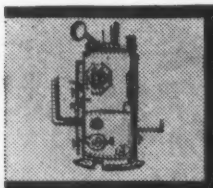

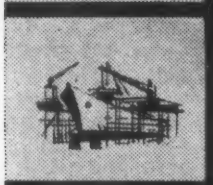

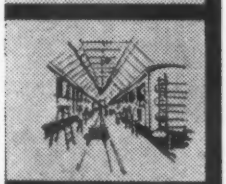

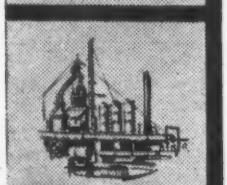

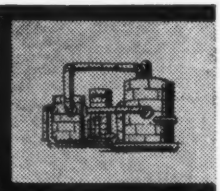

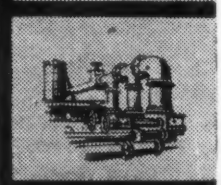

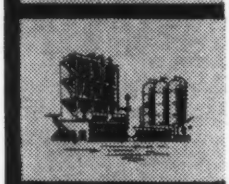

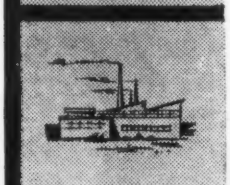

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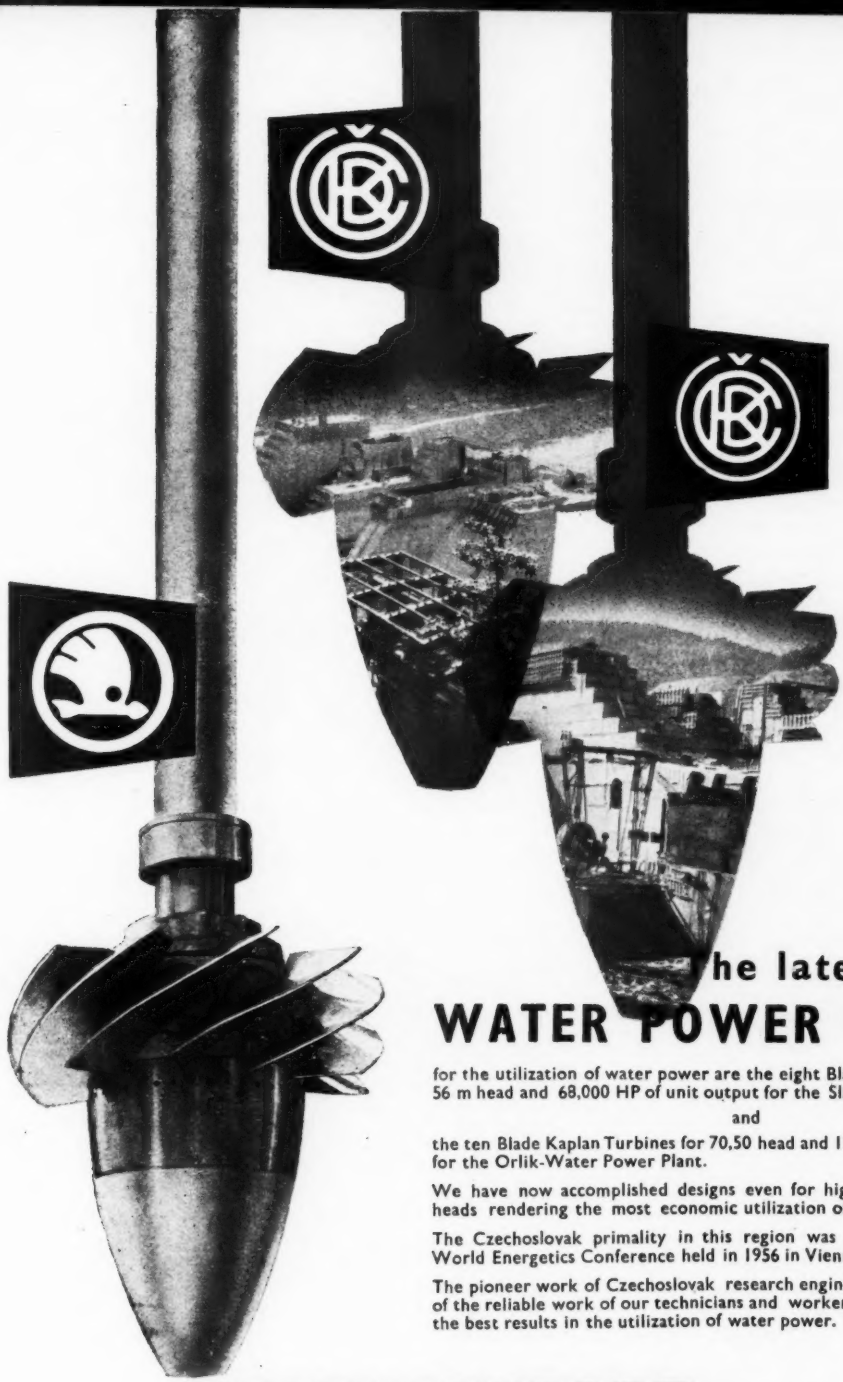

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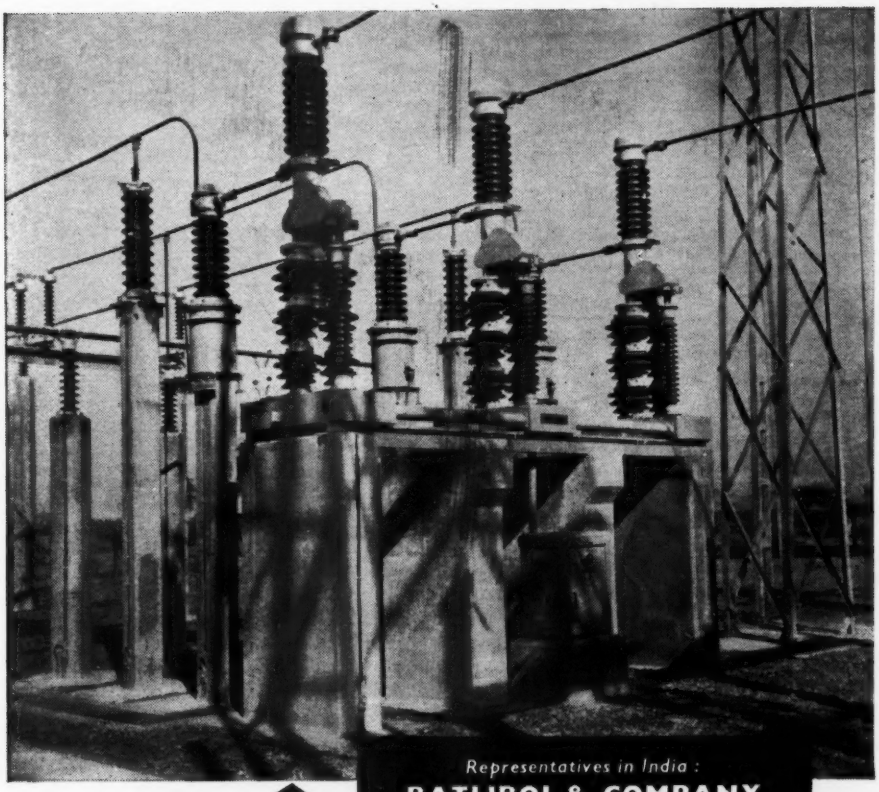

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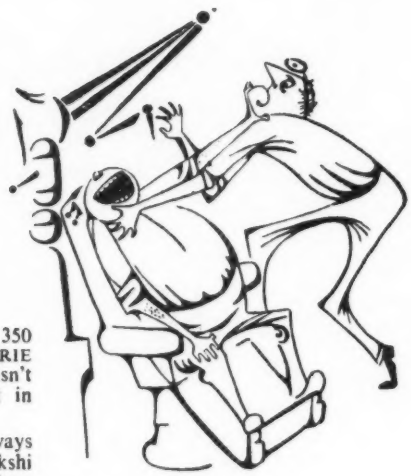

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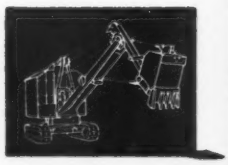

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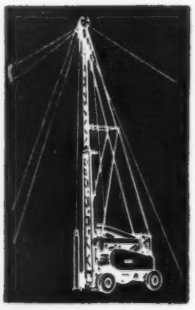

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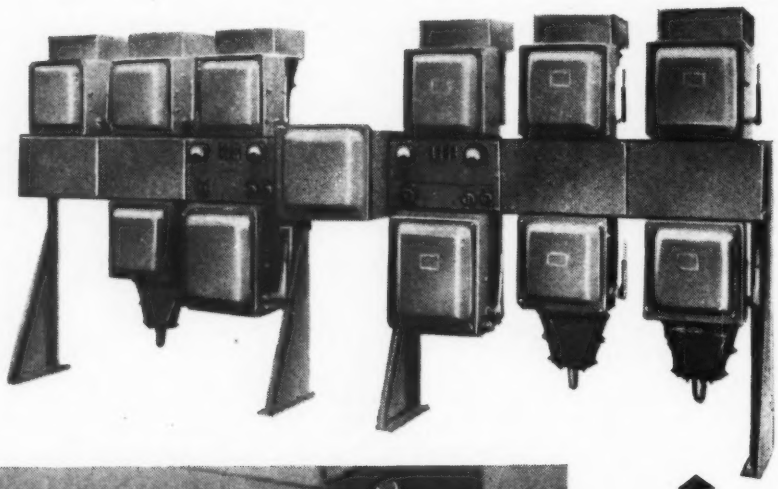

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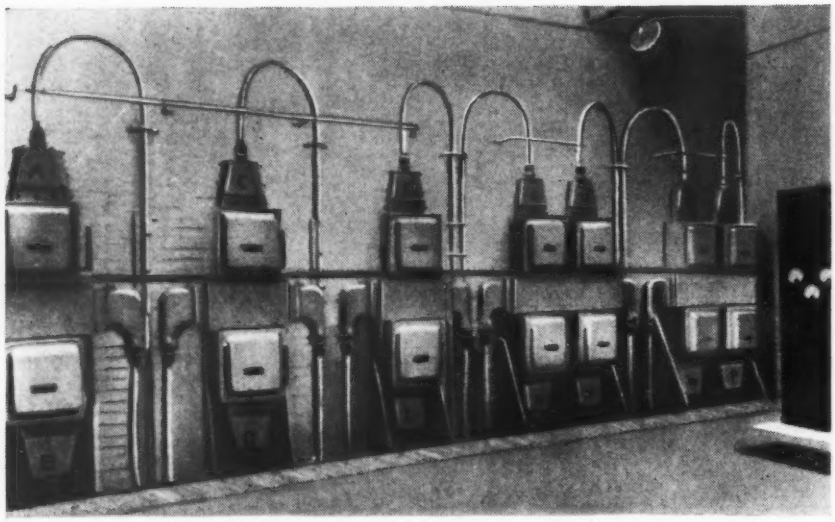

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Kariba Hydro-Electric Scheme

DR. B. K. R. PRASAD

This is a new hydro-electric project on a 1700 miles long Zambezi river in Africa situated at about 100 miles North-west of Salisbury at the boundary between the North and South Rhodesia. It is 250 miles downstream of Livingstone and Victoria Falls and is centrally placed for distribution of electric power to the copper belt of Northern Rhodesia and the industrial development of Southern Rhodesia. The head available is about 310 ft. and the lake will be 175 miles long and 40 miles wide, covering 2000 sq. miles.

The power station will be 1500 mW of which the I stage is 600 mW (6 × 100 mW) and II stage 900 mW.

At another site between Kariba and the Falls offers a possibility to develop a head of over 500 ft. and at a third site in Mozambique where a larger block of power than at Kariba can be utilised. The dam at Kariba is of the arch type 40 ft. high to provide storage and necessary head for the power plant and incidentally regulate the flow of water completely. There will be two underground power stations, one on each bank of the river and close to the dam. Only very compact diversion will be necessary.

The I stage comprises the entire dam, a power station on the right bank of 600 mW (6 × 100 mW). Two sets are now installed and the rest will be installed by 1962.

The II stage consists of the left Bank Power House of a capacity 900 mW to be ready to 1962. This will be extended by 300 mW to the final of 1200 mW. Thus the total capacity of the project will be 1500 mW and ultimately 1800 mW. The total annual output will be 8500 mkWhr. The development to the ultimate capacity will depend upon the load factor of the network for I and II phase, or the peak load. When the operation of the reservoir is fully achieved the total output will be known. The spillway is controlled by 6 sluice gates below the lowest operating level. Anti-scour arrangements are not necessary as the depth of the tail water gives adequate cushioning. The jet from the sluices also thrown the water clear off the dam.

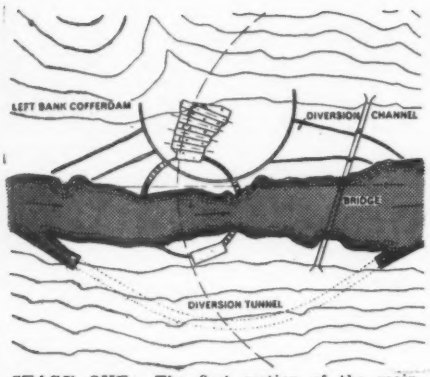
The power house of the I stage is laid out in the conventional manner underground with the transformers in a separate Hall. The draft tube gates are linked with the penstocks in the same shaft. There are surge chambers on the downstreams side of the tail water, since there is back pressure on the turbines.

The installed capacity is big enough to meet the maximum demand which entails close regulation of the construction programme. The cost involved is high and had to be incurred in any case. A very large transmission system is involved.

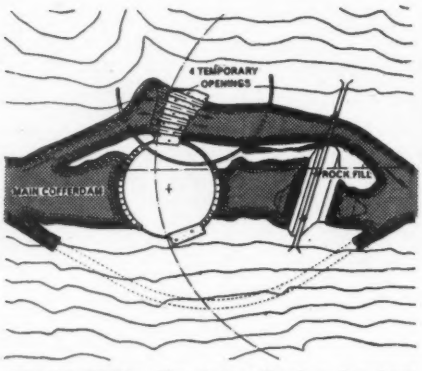
The site selected for the dam is 2 miles from the upstreams end of the gorge resulting in a dam whose length at the road level (1606 ASL) is 2025 ft. and height above foundations 420 ft. The rock in the gorge is a gneissic mountain mass, with dykes of granitic pegmatite cutting across the gneiss in various directions but playing only a subordinate part. The grey gneiss is by far the predominant rock. Although the right bank is covered with quartzite above approximate level 1450 ASL. Quartzite is a hard rock but very jointed. Gneiss is generally sound and monolithic. In some sections the rock is weathered at the left bank to depths of over 100 ft. Considered overall, the conditions for foundations were favourable fo a major arch dam. In the underground power house section gneiss varied in quality with some foliation and zones of altered rock and the gneiss is impervious. At the time of low flow the river was confined to a narrow channel, thus facilitating the sequence of coffer dam construction.

The arch dam is of double curvature with a large radius. (Height : crest length : 1 : 5). The thickness at the crest is the same as at the toe due to the long span. Max. thickness is 80 ft. (Ht. 420 ft. above foundation). There is a 40 ft. carriage way carried over the crest of 2025 ft. length. Six flood gates are the only hydraulic work built into the dam and there is no scour culvert. The water in the reservoir will not be drawn down below 1500 ft. level. Reasons for the arch dam : The design width was a minimum at foundation level thus reducing to a minimum the excavation in the difficult conditions at river level and also of the weathered rock on the flank. As the dam has a high centre of gravity the quantity of concrete at the lower levels was reduced, thus allowing fast progress and an early commencement of filling the reservoir. The dam is comparatively insensitive to overtopping and direct spillage over the crest level would have been possible during construction. The design in a "cupola" form results in a minimum volume of concrete for comparable concrete stresses. The optimum curvature in the vertical direction (upstream overhang in the lower part of the dam and a downstream overhang at the crest) was greatly reduced.

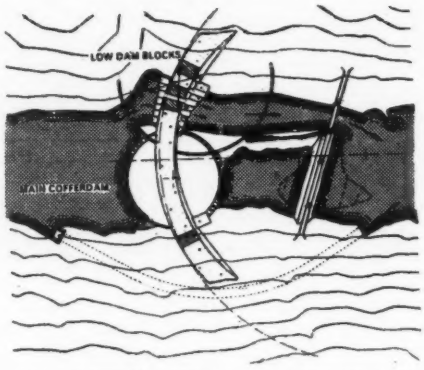
Model studies were made on 1/400 scale and stresses were studied under loads. There will be no "reservoir empty" condition and the correspond-



STAGE ONE: The first section of the main dam wall is built within the first coffer-dam while part of the river is channelled through a diversion tunnel.



STAGE TWO: The first coffer-dam has been breached, and a second temporary dam built. The Zambezi now flows through the diversion channel.



STAGE THREE: The dam wall is built in the main coffer-dam. Diversion channel and tunnel are in use, but soon the channel will be sealed.

ing load will never be experienced. The max. compressive stress will nowhere exceed 1000 lb/sq. in. but approaches this figure throughout the body of the dam. Work was started in November 6, 1956 and completed on June 22, 1959. About 1275000 cu. yds. of concrete were placed in 31½ months. Batching and mixing of concrete were carried out in 2 similar stations on the left bank with four 4 cu. yd. mixers. The skips were of 8 cu. yds. Concrete was placed at intervals of 50 ft. in 7 ft. 6″ lifts. A refrigerator plant with a capacity of 10,000,000 B Ther per hour was installed to cool the river water and the refrigerant was circulated until the temperature inside the concrete dropped to 77°F. The lifts were poured at 7 day intervals.

The cement content for the mass of the dam is 330 lb/cyd and for a thickness of 10 ft. on the water face it is 430 lb/cu. yd. Cement as per BSS 12 is used and the water cement ratio is 0.5 to 0.6. River sand was used and the agreegate was of crushed gneiss ranging from 8 to 1. Immersion vibrators 47 kg Poker length 24″ frequency of vibration 8700 rpm and no air entraining agents were used. Min. crushing strength was 3200 lbs. per sq. in at end of 28 days. An average cube strength of 4270 lb/sq. in. was obtained. At peak period 360 cu. yds/hr. were produced and monthly about 100,000 cu. yds. Cement came from 125 miles and was kept in silos. Grout curtain was filled between 80 ft. and 200 ft. below foundation. Six flood gates each 30 ft. high × 31 ft. wide are fixed with sill level at 1498 ft. One set of stop beams is provided for maintenance. One sluice is designed for passing debris. Discharge from all six gates at max. water level is 336,000 cusecs.

POWER STATION

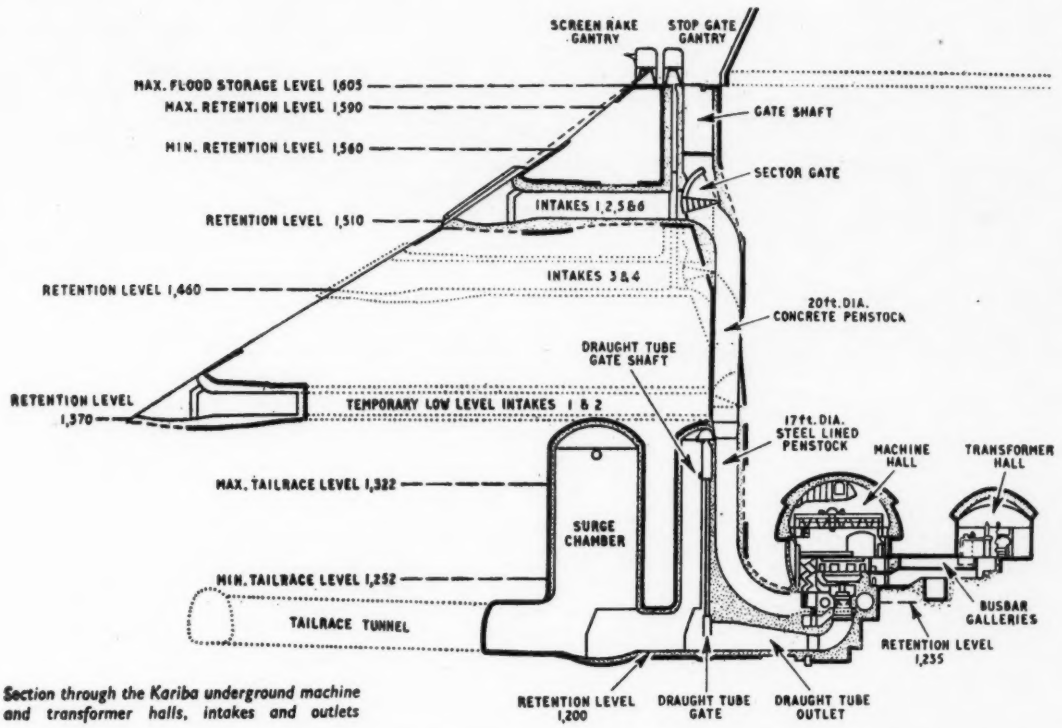
The underground machine hall will accommodate six sets of 100 mW turbo alternator sets. It is 465 ft. long × 75 ft. wide × 132 ft. high. A ramped tunnel leads from the surface to the machine hall. There are 2 lifts which ascend to the control building on the surface—two-200 ton overhead cranes serve the power plant in the machine hall. All heavy machinery are brought through the access tunnel to a loading bay.

Each turbine has a separate intake and penstock, the intakes being located on the side of the gorge with screens which can be raked from a gantry. The horizontal intake tunnels are each 16 ft. wide × 20 ft. high and are controlled by a sector gate placed immediately upstream of the 90° bend into the penstock which are 20 ft. in diameter and converge to 17 ft. in the steel lined section carried to level 1364.

For sets No. 1 and 2 low level intakes at 1370 ft. level have been provided to facilitate their commissioning at an earlier date before the reservoir fills up. Sets 3 and 4 have intakes at level 1460 ft. For sets 5 and 6 the intakes are at level 1510 ft. Intakes were designed on the basis of model experiments. The draft tube outlets are on the same side as the penstocks and after the gates they are paired into 3-tail race tunnels of equivalent 34 ft. diameter and are mostly unlined. There is a surge chamber of 63 ft. dia. at the head of each tunnel. The transformer hall is parallel to the machine hall and is 537 ft. long × 55 ft. wide × 60 ft. high. Three transformer banks of 240 MVA (3 single phase transformers) are installed to step up the power from the generating voltage of 18 kV to 330 kV. Power from the high tension terminals is taken by oil filled cables up a vertical shaft to the switching station on the surface on top of the gorge.

POWER STATION PLANT

The design of the plant is very simple. On account of the heavy weights and large dimensions the generators were shipped in quarter sections and reassembled at site. The turbine runners are 13 ft. 6″ dia. and weigh 40 tons each. They are of cast

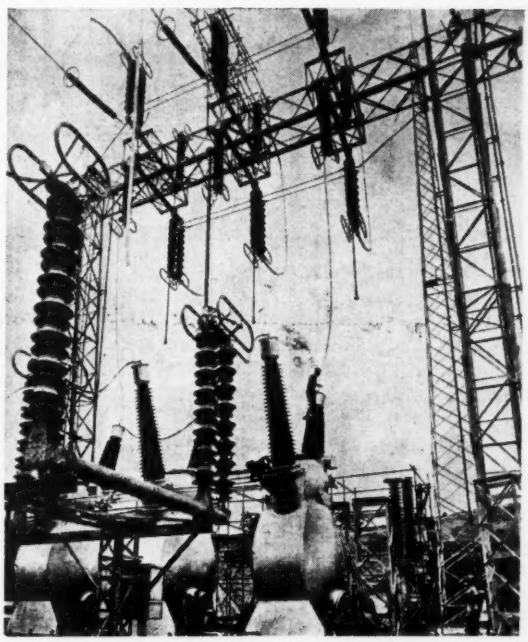


Section through the Kariba underground machine and transformer halls, intakes and outlets

and fabricated construction and have 18 stainless steel blades. The turbines are of vertical shaft Francis type and built after scale model studies regarding cavitation. The generators are directly coupled to turbines and run at 167 r.p.m. They are umbrella type—rotor mounted above the guide and thrust bearings, which are water cooled. The generator circulating air is also water cooled.

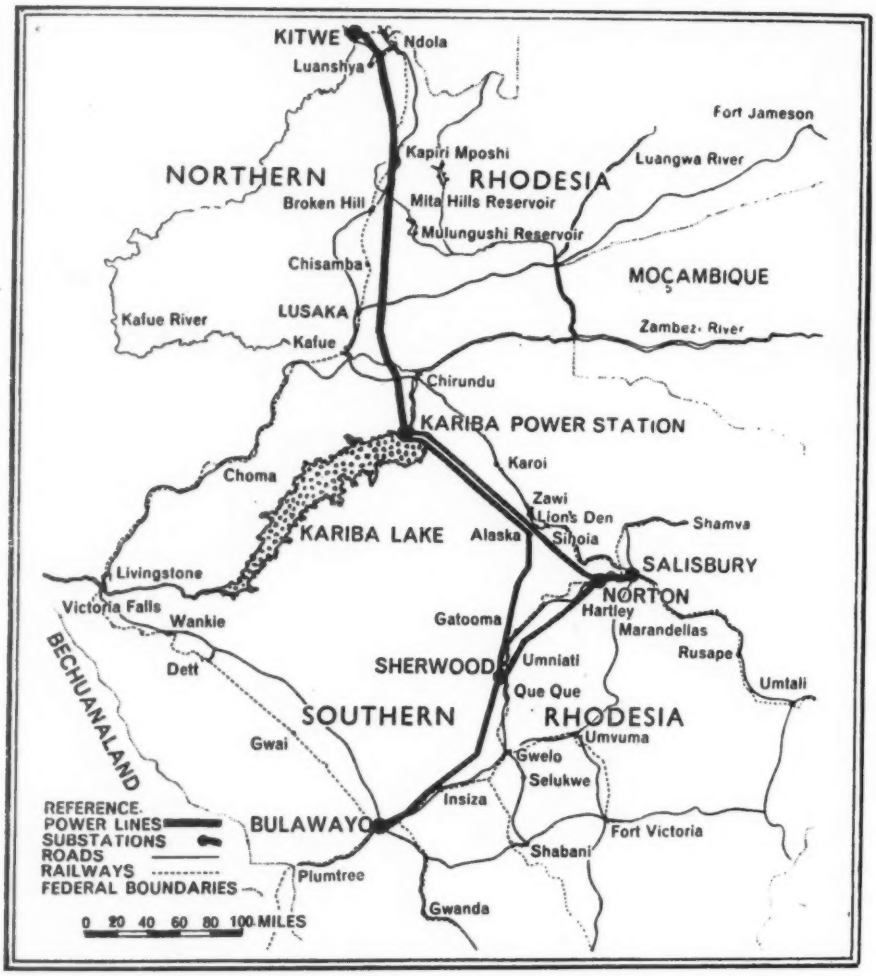
The generators are of 100 mW (111 MVA) 18 KV—3 phase 50 cycles. 0..9 P.F. exciters are of 364 kW, 360 volts D.C. direct coupled to the generators. Regulation is effected by means of rotating and static amplifiers. The auxiliary plant includes built in fire fighting equipment initiated by thermostats in the hot air circuit, a high pressure oil operated rotor jacking system, governors and bearing oil purifying equipment and an air compressor unit for braking. The machines are designed for a normal speed rise of 21%. When full load is suddenly thrown off, the run away speed with guide vanes fully open and no load on generator is nearly double the normal speed.

The transformer hall houses the generator transformers, station transformers surged diverters and station transformers switchgear. The banks of 3 single phase transformers are of 240 MVA (equivalent to 2 generators) with one primary winding of 18 KV for each generator. A common 330 KV winding delivers power by 330 KV to the Switching Station at the surface and thence to the transmission



Switchgear apparatus at Kariba.

a stem, where 330 KV oil circuit breakers of 7500 MVA fault capacity control each pair of generators and each outgoing feeder.



The Grid in Northern and Southern Rhodesia.

There is no tap changing on the transformers as the regulation is done by the generator excitation system. The transformers are water cooled (from water of turbine casings).

Reactors of 20 MVA are installed in certain places on the transmission system but they will be replaced by synchronous compensators as the load increases. The generator switchgear between the generators and transformers are of high current and high short circuit rating—4000 amp. 1500 MVA air blast type with operating time 0.03 second and total breaking time 0.05 second. Hollow section copper bus bars are used to connect the generators to the transformers.

Surge diverters are connected to the 330 KV terminals of the transformer and they discharge surge currents to the ground then the voltage at the transformer terminals exceeds a certain specified value, due to lightning.

The oil filled cables of 330 KV are in lengths of 660 yds. and are designed for a difference in level of 600 ft. (250 lb/sq. in.) hydrostatic pressure of oil. The 330 KV oil circuit breakers are of the bulk oil lenticular tank type having 6 breaks per phase.

The data on the dam, reservoir and power plant are shown in the appendix. The demand for electrical energy is expected to be 3200 million kWh in 1960 and 7200 million kWhr by 1970. The transmission systm will consist of a 330 KV line 267 miles long, northwards to tie with Belgian Congo power system, one line south of 183 miles to Salisbury, one line south of 327 miles to Bulawayo. Some of the transmission lines will run at an elevation of 5000 ft. Basic insulation level will be 15000 KV.

The steel towers will be 115 ft. high with twin peaks for 2 earth wires—counterpoise earthing will be used when footing resistance is 20 ohms. Twin conductors are used for each phase and they are ACSR— 54/0.118 al. and 7/0.118″ steel by British Insulated Cable Construction Co. and Richard Johnson and nephew. The phase clearance is 37 ft. 5″ minimum ground clearance is 25 ft. at 150°C. The standard span is 1500 ft. Insulators are of glass by Pilkington Bros. 19 discs 10″ dia. 5¾ spacing. Wet flash over for one min. is 600 KV. Tension sets have 18 insulators--11″ dia. Three zone high speed distance protection is provided. Mho type relays are used.

Control room for all switchgear are situated on the surface above the generating station—system control is arranged at a distance of 200 miles from Kariba. Telemetering system is provided. The approximate cost of I stage is about £ 79,400,000 i.e. 132/kW for 600 mW. The total cost of the I and II stage will be £ 119,400,000 for 1500 MV and works out at £280 per kW. The above include the cost of transmission system. About £ 39,000,000 of first stage are for civil engineering works. The cost of the plant and transmission equipment forms ⅓ the total cost and the balance is the cost of resettling the Africans, administration and other financial charges :

Sources of funds :

£ 28,500,000 for I stage--World Bank.
£ 20,000,000 „ —Copper Industry N. Rhodesia.
£ 4,000,000 Br. South African Co.
£ 4,000,000 Private Banks in the Federation.

One Generator set was commissioned on	Dec. 28, 1959.
Second Generator „ „	May 13, 1960.
Two more sets will go into service in	May 1961.
Two more sets „ „ „ „	1962.

The entire project will be completed between 1967 and 1972.

In the seven sub-stations there will be 25 breaks of 330 KV and 7500 MVA capacity and 100 isolators of 330 KV. on three phase reclosing duty. The breakers are of bulk oil lenticular type with six breaks on each phase and each break is shunted by a linear resistance and is pneumatically operated. The transformer at sub-stations are of 3 phase type

2 Transformers
120 MVA 330/234 KV 3 p. auto transformer.
8 Transformers
60 MVA 330/88 KV 3 p. double wound.
2 Transformers
60 MVA 330/33 KV 3 p. „ „

Summation Meters have been installed in power stations.

CONSULTANTS AND CONTRACTORS

Consultants :

Civil Engineering Works	: Gibb, Coyne, Sogeie (Kariba Private Ltd.)
Mechanical & Electrical Works	: Merz and McLellan.

Contractors :

Turbines	: Boving & Co. Ltd.
Generators	: A.E.I. Export Ltd. (Metropolitan Vickers).
18 KV Switchgear for Generators	: Brown Boveri Ltd., Manchester.
Generator Transformer	: English Electric Co. Ltd.
330 KV Switchgear	: A.E.I. Export Ltd. (B.T.H. Co.) Willesden.
Sub-station Transformers	: Ferranti Ltd.
Overhead Travelling Cranes	: Babcock & Wilcox Ltd.
Shunt Compensating Reactors	: C. A. Parsons Ltd.
330 KV Oil Filled cables	: British Insulated Callender Cable Co. Ltd.
Other Cables	: Siemens Edison Swan Ltd.
Transmission Lines	: Impredit (Italian Consortium).

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PRINCIPAL DATA FOR THE KARIBA SCHEME

Installed capacity Right Bank	:	600 mW (ultimate).
,, ,, ,, ,,	:	200 mW (present).
,, ,, Left Bank	:	900 mW to 1200 mW.
Ultimate mean annual power output	:	8500 million kWhr.

Dam :

Max. height of dam	:	425 ft.
Crest length of dam	:	2025 ft.
Mean radius of arch	:	800 ft.
Total concrete Volume	:	1275000 cu. yds.
Spillway gates number	:	Six.
,, ,, size each	:	30 ft. × 31 ft.

Reservoir :

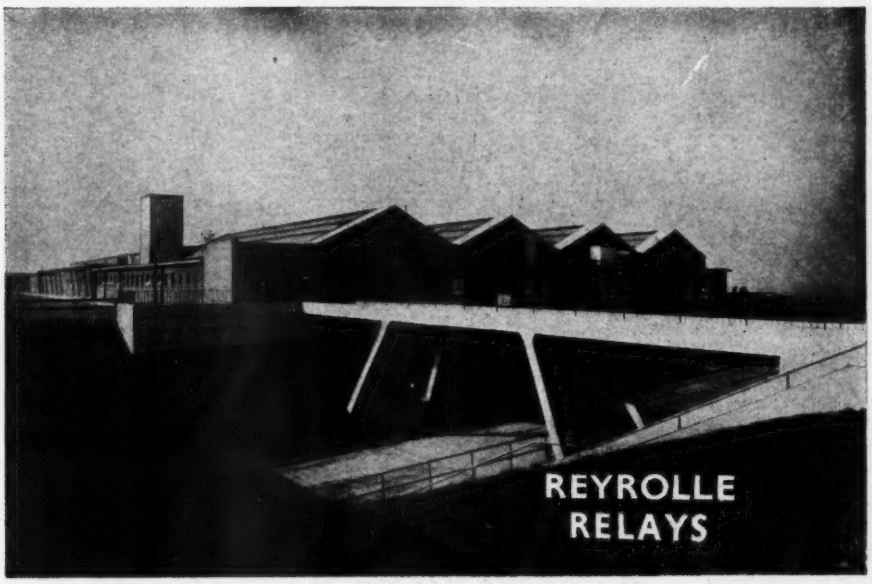
Reservoir capacity at 1605 max. flood level	:	149×10^6 acre ft.
,, ,, at 1590 ft. normal water level	:	130×10^6 acre ft.
Normal operating range of water levels	:	1560–1590 A.S.L.
Useful storage capacity within normal range of water levels	:	36 million acre ft.
Flood storage	:	19×10^6 acre ft.
Surface area of Reservoir	:	2000 sq. miles.
Mean River flow	:	51000 cusecs.
Total mean flow in one year	:	37 million acre ft.
Utilisation for power	:	83%.
Losses by spillage	:	8%.
Losses by evaporation	:	9%.
Firm flow for power	:	42000 Cusecs.
,, ,, in one year	:	30 :5 million acre ft.
Estimated probability of occurrence of mean monthly run offs		
1 in 100	:	20 million acre ft.
1 in 1000	:	27.5 ,, ,, ,,
1 in 10000	:	34.6 ,, ,, ,,
Capacity of Sluice gate installation		
At 1590 Top water level	:	306000 cusecs.
At 1605 Top water level	:	336000 cusecs.
Minimum Tail water level 10000 cusecs	:	1253 A.S.L.
Tail water level at meon flow for 41000 cusecs	:	1267 A.S.L.
Max. Tail water level 360,000 Cusecs	:	1325 A.S.I.
Range of gross heads	:	352 ft. to 265 ft.

Quantity Data for Construction I Stage Works:

	Excavation.	*Concrete.*	*Steel.*
For Dam	500,000 cu. yds.	1,275,000 cu. yds.	3,700 tons.
For power station Machine and Transformer Halls	240,000 cu. yds.	70,000 cu. yds.	1,250 tons.
For Intake Penstock Shafts & Tunnels	515,000	120,000	6,000
	1,255,000	1,465,000	10,950

Power Station :

Dimensions of Machine Hall	:	468 ft. long × 75 ft. wide × 132 ft high.
,, Transformer Hall	:	537 ft. long × 55 ft. wide × 60 ft. high.
Turbines : Max. gross head	:	352 ft.
Mean net head	:	310 ft.
Full load output at 282 ft Net head	:	140000 bhp.
Corresponding rated flow through turbine	:	4750 cusecs.
Total Weight of each alternator	:	720 tons.
No. of alternators	:	6 of 100 mW @ 0.9 PF 18 KV 50 cycles.
No. of single phase transformers	:	10 of 80 MVA 18/330 KV.
Dia. of alternator Rotor	:	25 ft. 6″.
Dia. of Turbine Rotor	:	13 ft. 6″.
Wt. of Turbine Runner	:	40 tons.
Speed of Turbine and alternator	:	167 rpm.
Wt. of rotating parts of turbine & generator	:	475 tons.
Total load on thrust bearing (Wt. of Rotors plus hydraulic thrust)	:	660 tons.
Overall diameter of alternator	:	32 ft.



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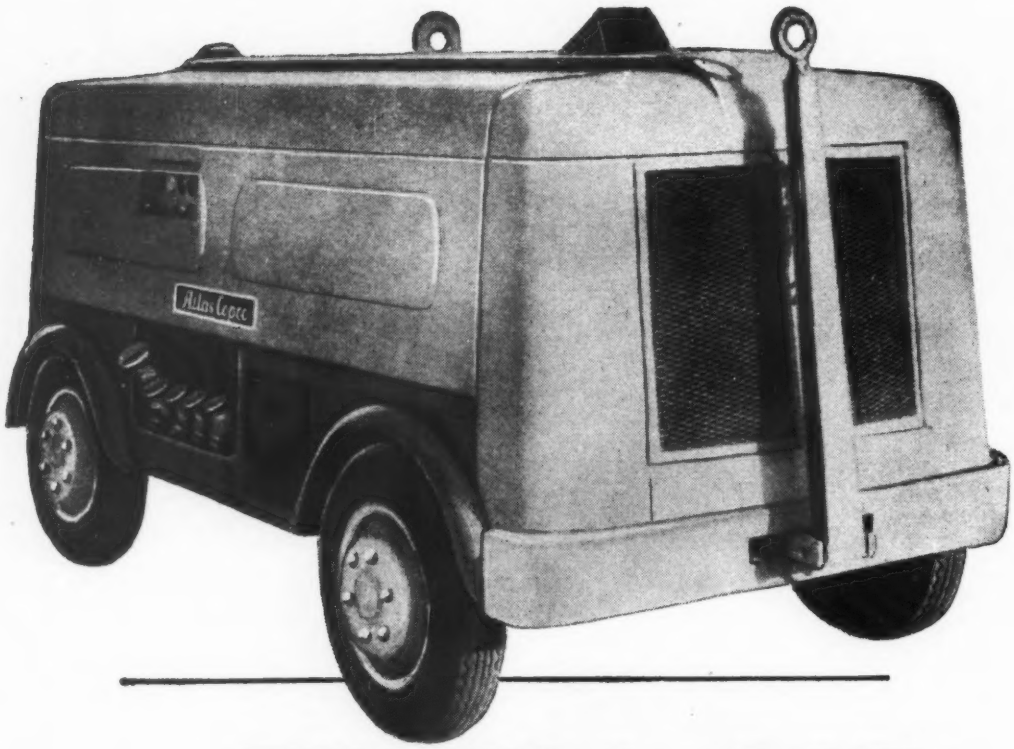

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The Kariba Dam Opened

Queen Elizabeth the Queen Mother on May 17 opened officially the vast Kariba hydro-electric project, which harnesses the River Zambezi to produce power for a large area of Central Africa.

The Paymaster-General, Lord Mills, represented the British Government at the inauguration of the project, which had already cost about £ 75,000,000. Kariba, Africa's biggest hydro-electric scheme and one of the great civil-engineering feats of the century, is expected to cost a total of about £120,000,000.

At the opening ceremony, the Queen Mother said : "I pray with all my heart that the Kariba dam may be a symbol of new and wider understanding throughout the Federation of Rhodesia and Nyasaland and, indeed, throughout this mighty continent of Africa." This massive source of energy, by raising the living standards of the people, would bring untold blessings to future generations.

"MARVEL OF MODERN ENGINEERING"

The Federation, she said, had created in the heart of Africa a "marvel of modern engineering which, in the future, may well rank with the Seven Wonders of the ancient world".

For nearly five years, 7,000 Africans and 1,500 Europeans have laboured on the project.

With the completion of the Kariba project, which feeds power to the Northern Rhodesian Copperbelt and Southern Rhodesia's fast-growing industry, another part of the great continent has been opened to modern progress.

The project, with its 420 ft.-high dam and underground powerhouse, is creating the world's biggest man-made lake, 180 miles long and, in places, 40 miles wide.

The rocky Kariba gorge forms part of the border between Northern and Southern Rhodesia. Through it, only a few years ago, the river Zambesi flowed unimpeded—a huge river even in the dry season, growing to a foaming flood after the annual rains. Now it is held by a great curved wall of concrete—the Kariba dam, one of the modern engineering wonders of the world.

The water is rising gradually against the face of the dam and spreading over the low-lying country above the gorge. Eventually it will form a lake that stretches 175 miles upstream—the largest man-made reservoir in the world. The power of this water will be used to produce electricity for the people of Northern and Southern Rhodesia and for the growing industries of the area.

The Government of the Federation of Rhodesia and Nyasaland realized some years ago that the demand for electricity was growing fast and would soon be too great for the coal-burning power stations that the country already possessed. After very careful investigations it was decided to undertake a hydro-electricity scheme at Kariba.

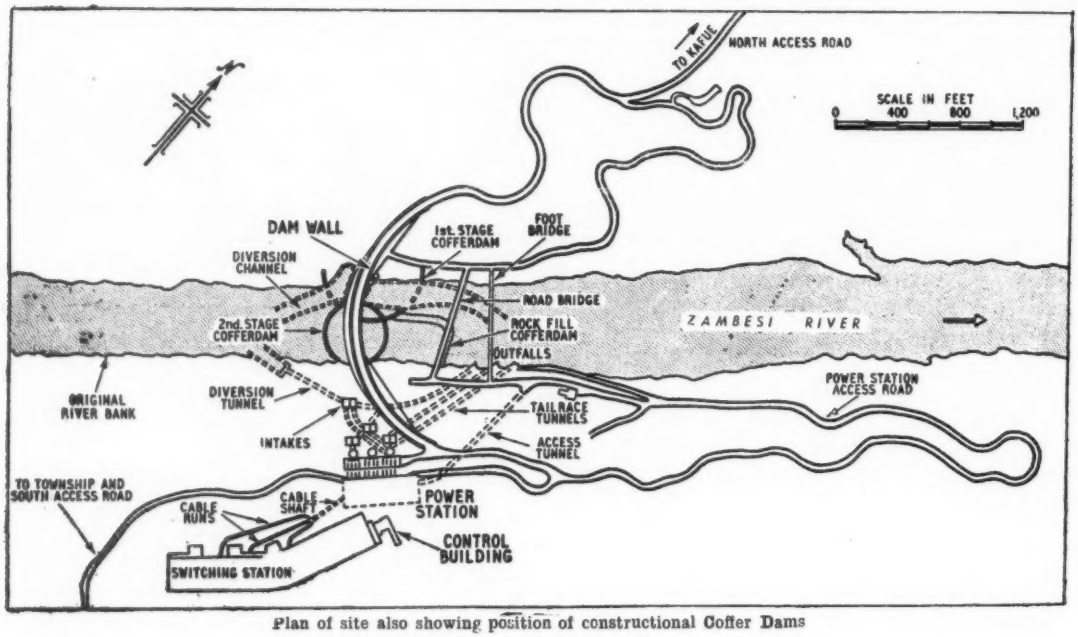
KARIBA MILESTONES

March, 1955	: Decision to proceed with Kariba is announced.
July, 1955	: Contracts awarded to firms for preliminary works.
July, 1956	: Main Civil engineering contract awarded. Contracts placed for power station equipment.
February, 1957	: Floods.
July, 1957	: Diversion of river and start of main coffer-dam.
November, 1957	: Main coffer-dam dewatered. River bed exposed.
Feb./March, 1958	: Exceptional floods. Main coffer-dam overtopped.
June, 1958	: Main coffer-dam dewatered after floods.
December, 1958	: Dam wall closed, impounding of Kariba Lake commenced.
June, 1959	: Main concreting completed, except for roadway.
January, 1960	: The first electric power is generated at Kariba.
May, 1960	: Official opening of Kariba by Her Majesty the Queen Mother.

WORK BEGINS

The Federal Hydro-Electric Board, now in charge of the project, needed expert advisers to design the works and to supervise construction. In June 1955, three firms—Sir Alexander Gibb & Partners, of London ; Coyne & Bellier, of Paris ; and the Societe Generale d'Exploitations Industrielles—were nominated as civil engineering consultant and asked to work as a team. They at once opened an office in Salisbury, the capital of Southern Rhodesia. Another firm, Merz & McLellan, was appointed to advise on all questions of mechanical and electrical equipment. Within a month, the first contract, for river diversion works, had been awarded.

Meanwhile, detail design work for the dam and power station was being pressed ahead in London



Plan of site also showing position of constructional Coffer Dams

and Paris, access to the remote site was being opened up, and a new town was being constructed to house the thousands of workers on the job. Almost exactly a year later, the main civil engineering contract, some £25,000,000 in value, was awarded.

A working programme covering several years was drawn up at the start. This was arranged so as to fit in with the river's annual periods of high and low flow.

STORY OF CONSTRUCTION

A temporary dam or cofferdam was built out from the left bank of the river, enclosing a semi-circular area within which the first section of the main dam was constructed. Then parts of the coffer dam were blown up, and construction of the rest of the dam was started within a second cofferdam much larger than the first. The river was diverted partly through a tunnel excavated in the right bank and partly through temporary openings in the first section of the dam.

When the dam had reached a height of about 250 feet, the diversion tunnel and the temporary openings were permanently stopped up. Only a little water was allowed to pass through, so that the river downstream should not dry up completely. The filling of Kariba Lake had started. From that time onwards, construction of the dam had to be carried on at such a speed that the top was always above the rising water.

Meanwhile, a powerhouse was being prepared to accommodate the water turbines and generators. This is an underground chamber 468 ft. long. Alongside it is another chamber, even longer but not so wide, for the transformers. There are tunnels and shafts to deliver water from the reservoir to the turbines beneath the powerhouse floor, and other tunnels to discharge the used water into the river below the dam. All these were excavated out of solid rock.

The township was built by Richard Costain (Southern Rhodesia) Ltd.; the diversion works were entrusted to the Cementation Co. Ltd., and the access roads and airfield to a third Brittish firm, John Laing & Son (Rhodesia) Ltd. The main contract, including construction of the dam itself, the powerhouse and other underground works, was opend to international tender and awarded to a group of Italian firms, which obtained such of their equipment and materials from Britain and South Africa. British companies, which won the contracts in open competition, are supplying turbines, generators, and other electrical equipment.

THE ZAMBESI FLOODS

In the spring of 1957, following heavy rains, the annual floods rose as high as they had ever done since records had been kept. The river came over the top of the first cofferdam and filled it. Next year the floods were much greater still. The second cofferdam—a circular wall 160 feet high across the centre of the river—disappeared beneath the rushing water.

These emergencies had been provided for, and no permanent damage was done. But the flooding of the works delayed construction for several weeks. Each time, however, by a great effort the lost time was soon made up. By June 1959, the last bucket

of concrete had been placed on the top of the dam many months ahead of schedule, and plant installation was well in hand.

The programme drawn up in 1955 has been adhered to, in spite of all difficulties, with the starting of the first turbine, and as demand increases five more turbines and generators will come into operation. Work has started on a second powerhouse on the left bank. When this is complete, Kariba will produce an average of 8,500,000,000 kilowatt-hours per year.

COMMUNICATION CHANNELS

To distribute this power from Kariba expeditiously, over an area stretching between Kitwe in the copper belt 267 miles to the north, Salisbury 183 miles to the south-east, and Bulawayo 327 miles to the south-west, a System Control Centre (S.C.C.) has been established at Sherwood, about 200 miles due south of Kariba, and approximately midway between Bulawayo and Salisbury. This S.C.C. has been provided with some of the latest and most comprehensive control and communication equipment available anywhere in the world. This article describes briefly the techniques used to solve the problem of economical system operation over such vast distances.

The main power transmission network operates at 330 KV. Superimposed on this network is anything from 1-6 power line carrier channels per feeder for control and communication purposes, each channel with a band width of 4 kc/s. The carrier frequencies lie between the extremes of 84—458 kc/s.

Each power line carrier channel consists of a speech circuit and up to 6 voice frequency telegraph (VFT) channels. The speech band is 300-2350 cycles per second, and above this speech band there are 6 VFT channels operating at 120 cycles per second spacing over the range 2520-3240 cycles per second. The VFT operates on a frequency shift basis with plus and minus 30 cycles per second shift on nominal mid-band frequency. The choice of this system leads to a marked reduction in interference from noise in what is undoubtedly an area of high thunderstorm activity.

The VFT channels are allocated among :— (1) Supervisory indications and control, (2) Telemetering, (3) Teleprinter, (4) Telephone dialling, and (5) and (6) Spare.

SUPERVISORY INDICATION & CONTROL

The open or closed position of each 330 kV circuit breaker and isolator throughout the whole network is continuously supervised by means of the indication and control equipment located at Kariba and the seven switching stations. Each and every change of condition (whether intentional or unintentional) is signalled by means of a constant total train of pulses over the supervisory channel to sound an alarm bell in the System Control Centre and draw the attention of the operator to the part of the network under change by means of a flashing indication.

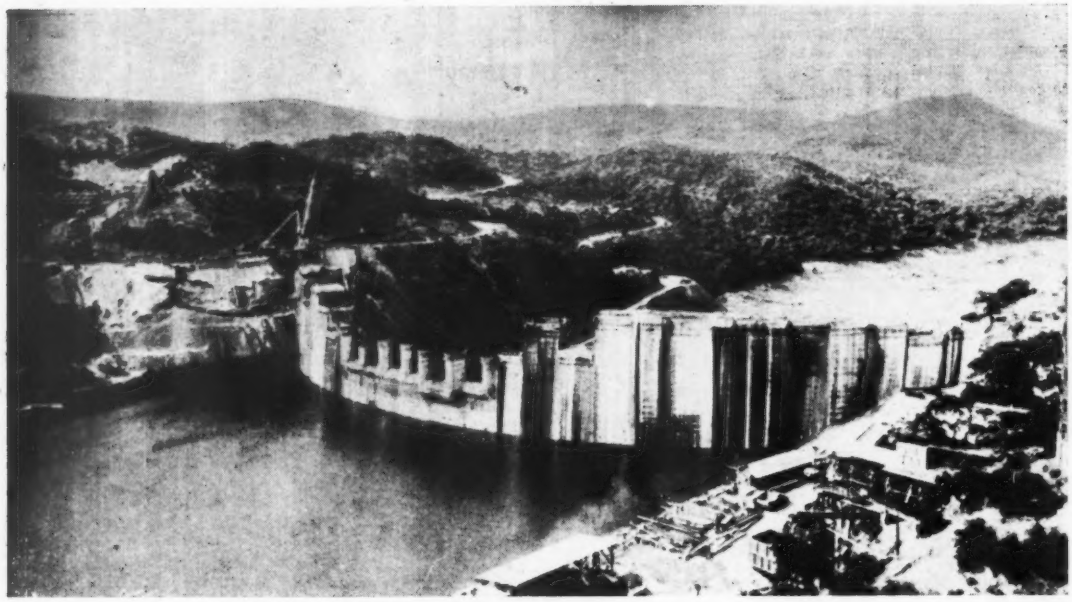
Construction work going on in the night at Kariba

At all times, therefore, the S.C.C. operator has a complete and up-to-date picture of the state of the whole network of nearly 900 circuit miles of 330 kV transmission line. The time taken to send a signal announcing a fault in a sub-station is little over two seconds for even the most distant station (Kitwe is 470 miles from the S.C.C.).

TELEMETERING

A vital part of the system control function is a knowledge both of the power available and of the power being generated at each power station. Equally is the system control operator required to know the load on the network at each of the "take-off" points. i.e. the 330 kV switching stations. This is done by measuring and transmitting kilowatt, reactive KVA and busbar voltage indications continuously over the VFT network. The Telemetering System (known as the Impulse Frequency System) is one of the most up-to-date in the world. Space does not permit detailed description of the technique, but suffice it to say that pulses are generated on integrating meters at the outstations so that the rate of pulsing is proportional to the power flow.

In order to achieve an economy in the transmission channels, a system of time-division multiplexing is adopted whereby up to six simultaneous read-



A panoramic view of the Dam under construction

ings are sent over any one VFT channel. The method of multiplexing, specially designed by Standard Telephones and Cables Ltd., is to allot to each reading a single teleprinter character. This character is generated electronically at the sending end, each letter corresponding to the pulse received from its associated initiating device. At the receiving end, the pulses are decoded and passed to the appropriate telemeters, where they are converted into steady readings which are displayed on meters on the 75-feet switching and loading diagram at the System Control Centre.

TELEPRINTERS AND TELEPHONY

A complete teleprinter network interconnects all the sub-stations with the S.C.C. Point-to-point communication between the S.C.C. and any outstation is given by means of a specially-designed concentrator located at the S.C.C., which also permits broadcast facilities from the S.C.C. to any or all outstations simultaneously.

Each of the seven sub-stations has a private automatic exchange of size varying from 25 to 50 lines. The whole network is interconnected so that trunk dialling is possible between any extension on any PAX and any other extention on the same or any other PAX. For local dialling within any one substation normal techniques are used, but for dialling between two PAX's connected by a power line carrier, line impulsing is carried out on one of the VFT channels. Priority facilities exist to enable any outstation to reach the System Control Centre even if junctions are engaged, and conversely any engaged extension can be interrupted by a priority call from the Sherwood control room. A specially-designed automatic keysender at the S.C.C. permits push-button calling from the control desk to selected extensions anywhere in the network.

PHOTO FACSIMILE EQUIPMENT

An additional refinement in the control and communication network is the provision of photo facsimile equipment enabling drawings, sketches or even photographs to be transmitted between the System Control Centre at Sherwood and the board's Head office at Salisbury, a distance of some 110 miles. The photo facsimile equipment uses an extra speech channel on the link from Salisbury to Sherwood via Norton, and pictures up to size 13×8 inches can be transmitted in under five minutes.

KARIBA POWER STATION

In addition to the comprehensive control and communication network provided between the various switching stations and the System Control Centre itself, the whole of the direct wire indicating, control, and alarm annunciation equipment for the Kariba Power Station and switching station is based on modern light-current techniques. This includes starting of the turbines, bringing up the speed, synchronising, connecting to the busbars and ultimate despatch into the 330 kV transmission grid. Power station control is effected from a compact desk backed by a mimic diagram indicating the switch conditions so that the operator has finger-tip control of all the machines in the power station, both now and for any future extension.

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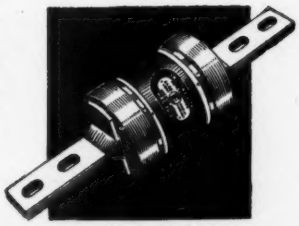
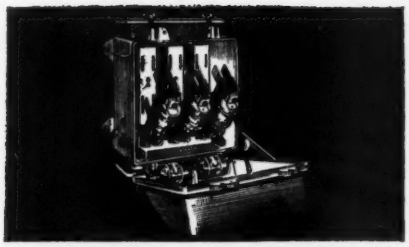
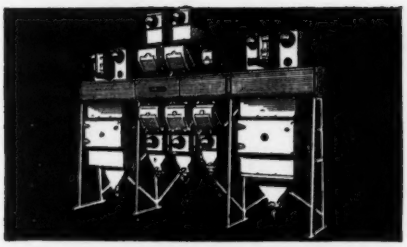




COMBINATION FUSE-SWITCHES
Ratings from 30 to 300 amps—Rugged construction, fully shrouded and interlocked—Type tested for 35 MVA rupturing capacity



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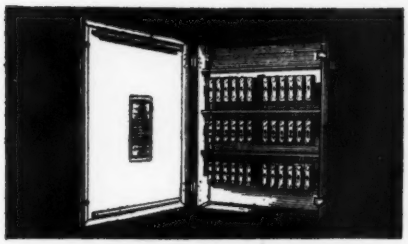

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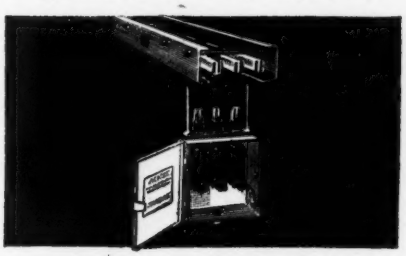

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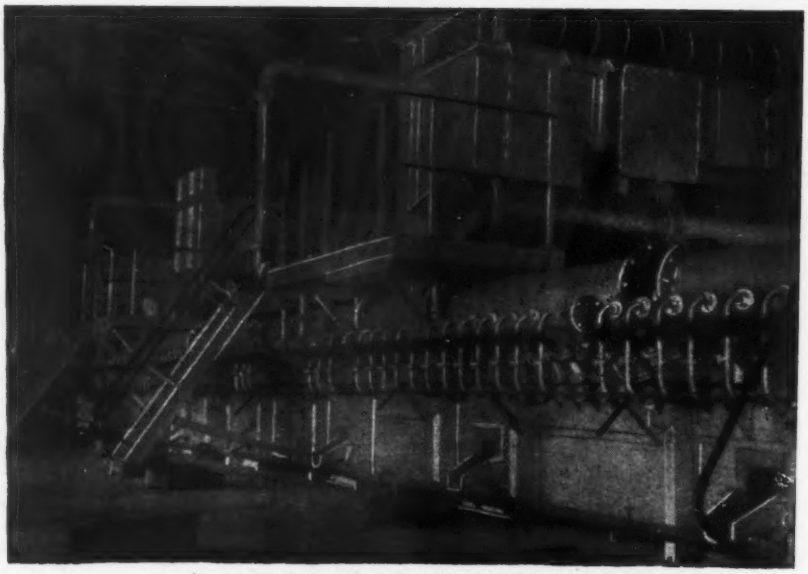

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Generalised Method for the Solution of Hydraulic Jump Problems in Rectangular Channels

By R. M. ADVANI

The problem of determining the elements of hydraulic jump is of vital importance when dealing with such hydraulic structures as barrages, weirs, canal falls, regulators, high spillway dams and so on since one of the most efficient ways of destroying the superfluous energy below such structures is by the formation of hydraulic jump of a suitable height and at a suitable place down stream. A large number of major River Valley Projects already under construction or in the advanced stages of planning, involve amongst other things, the construction of such hydraulic structures as mentioned above. In India the various methods in vogue for determining the elements of hydraulic jump are :—

(*i*) the Blench-Montague method,
(*ii*) the Crump's method and
(*iii*) the Brown's method.

Besides these, a few empirical formulae suggested earlier by Dr. Malhotra, Inglis, Joglekar and others have also been in use. These methods are not so suitable without some modifications when dealing with the problems of high spillway dams. This is probably due to the fact that high dams are the speciality of present days. Also these methods are not very general in application. The author has, therefore, attempted to furnish in this paper extended Crump's table and also a generalised table and generalised chart which it is hoped would be found to be very useful for determining the elements of hydraulic jump in rectangular channel particularly in the case of high spillway dams. The generalized table and the generalized chart would in addition be found to have much wider application as also advantage of exhibiting certain useful properties of the jump.

ELEMENTS OF HYDRAULIC JUMP

The various elements of Hydraulic Jump in a rectangular channel are :—

q, the discharge per ft. width.
D_1, the depth before the jump.
D_2, the depth after the jump.
E_1, the energy before the jump.
E_2, the energy after the jump.
H_L, the energy lost in the jump.

These six quantities characterizing the jump are connected by the following four equations :—

$$D_1\ D_2\ (D_1+D_2)=\frac{2q^2}{g} \quad \ldots\ldots \text{I (Momentum equation).}$$

This equation can be expressed in the various other forms as :—

$$D_2=\sqrt{\frac{2q^2}{gD_1}+\frac{D_1^2}{4}}-\frac{D_1}{2} \quad \ldots\ldots \text{I (a)}$$

$$=\frac{D_1}{2}\left\{-1+\sqrt{1+8\left[\frac{D_c}{D_1}\right]^3}\right\} \quad \ldots\ldots \text{I (b)}$$

$$=\frac{D_1}{2}\left\{-1+\sqrt{1+8F_1^2}\right\} \quad \ldots\ldots \text{I (c)}$$

Where D_c is the critical depth given by

$$D_c=\left(\frac{q^2}{g}\right)^{\frac{1}{3}}$$

F is the Froude's Number given by

$$F=\frac{V}{\sqrt{gD}}=\frac{q}{gD^{3/2}}=\left[\frac{D_c}{D}\right]^{3/2}$$

$$E_1=D_1+\frac{V_1^2}{2g}=D_1+\frac{q^2}{2g\,D_1^2} \quad \ldots\ldots (2)$$

$$E_2=D_2+\frac{V_2^2}{2g}=D_2+\frac{q^2}{2g\,D_2^2} \quad \ldots\ldots (3)$$

$$H_L=E_1-E_2=\frac{(D_2-D_1)^3}{4\,D_1\,D_2} \quad \ldots\ldots (4)$$

Thus theoretically given any two of the six quantities, the rest can be obtained analytically from these four relations.

In practice, however, it is difficult to obtain the solution analytically, particularly when the given elements are q and H_L as is usually the case. The various graphical methods in vogue are :—

(a) The Blench-Montague method.
(b) the Crump's method and
(c) the Brown's method.

Besides these, some empirical formulæ suggested earlier by Malhotra, Inglis, Joglekar and others have also been in use. These methods have been discussed at length by the author in his earlier paper.[2] The scope of this paper is to discuss in greater detail the author's generalized table for the solution of hydraulic jump problems and to compare it with dimensionless hydraulic jump co-efficient method given by Crump, Brown and others.

CRUMP'S DIMENSIONLESS HYDRAULIC JUMP CO-EFFICIENTS

The Crump's dimensionless hydraulic jump co-efficients ae :—

$$x=\frac{D_1}{D_c},\ y=\frac{D_2}{D_c},\ z=\frac{H_L}{D_c},\ m=\frac{E_1}{D_c}\text{ and }n=\frac{E_2}{D_c}$$

These are connected by the following four equations (obtained by substitution in 1, 2, 3 and 4) :—

$$xy(x+y)=2 \qquad \ldots.(5)$$

$$x+\frac{1}{2x^2}=m \qquad \ldots.(6)$$

$$y+\frac{1}{2y^2}=n \qquad \ldots.(7)$$

$$\frac{(y-x)^3}{4xy}=m-n=z \qquad \ldots..(8)$$

Extensive tables satisfying the above relationships have been prepared by the author[3] for the value of x from 0.01 to 0.30 at intervals of 0.001 from 0.31 to 0.70 at intervals of 0.01 and from 0.70 to 1.00 at intervals of 0.1. Also table for critical depths have been prepared by the author for values of q up to 1300 cusecs. The author feels that these tables are the most extensive of the type ever published before and give very accurate results without the use of curves, because of the calculated values at very small intervals of x and q. However, one of the given elements should necessarily be the discharge intensity q so as to be able to determine D_c. These tables are given here in part vide Appendix I.

Appendix 1

EXTENDED TABLE OF CRUMP'S DIMENSIONLESS HYDRAULIC JUMP CO-EFFICIENTS

x	y	m	n	z
0.010	14.1372	5000.01	14.1397	4985.87
0.011	13.4775	4132.21	13.4818	4118.73
0.012	12.9040	3472.21	12.9070	3459.30
0.013	12.3955	2956.61	12.3987	2944.21
0.014	11.9440	2551.01	11.9475	2540.16
0.015	11.5395	2222.24	11.5431	2210.70
0.145	3.642	23.93	3.679	20.25
0.146	3.629	23.55	3.667	19.89
0.147	3.616	23.29	3.650	19.64
0.148	3.602	22.98	3.640	19.34
0.149	3.590	22.67	3.630	19.04
0.150	3.577	22.37	3.620	18.75
0.65	1.459	1.837	1.694	0.143
0.66	1.442	1.811	1.682	0.129
0.67	1.424	1.789	1.671	0.118
0.68	1.409	1.764	1.661	0.103
0.69	1.392	1.741	1.650	0.091
0.70	1.376	1.7204	1.6405	0.0799
0.80	1.231	1.5813	1.5608	0.0205
0.90	1.1069	1.5173	1.5167	0.0019
1.00	1.0000	—	—	—

It may be mentioned here that Prof. Paylovsky also obtained the various dimensionless standing wave co-efficients by choosing critical depth, D_c, as the basis of reference though he expressed the relation between these elements in a slightly different form, from what was given by Crump. For example, he gave the relationship, $xy.x_m=1$ where $x_m=\frac{x+y}{2}$. It can be seen that this is similar to equation 5.

Further, Stevens and Woodward expressed the various dimensionless hydraulic jump co-efficients in terms of upstream depth D_1. Next Brown obtained the dimensionless co-efficients by choosing H_L as the basis of reference. His dimensionless co-efficients are $x_1=\frac{D_1}{H_L}$, $y_1=\frac{D_2}{H_L}$, $z_1=\frac{q}{H_L^{3/2}}$, $m_1=\frac{E_1}{H_L}$ and $n_1=\frac{E_2}{H_L}$. Also he expressed the relationship between these co-efficients in terms of one variable $\frac{D_1}{E_1}=D'_1$. Similarly Bakhmeteff[5] expressed the relationship between the various dimensionless co-efficients in terms of one variable, $\lambda_1=\frac{V_1^2}{gD_1}=F_1^2$.

The author has however generalized the method and expressed the relationship between various dimensionless co-efficients suggested originally by Crump, Brown, Bakhmeteff and others in terms of D'_1 only. This is obtained as under :—

Let $\frac{D_1}{E_1}=D'_1$, $\frac{D_2}{E_1}=D'_2$, $\frac{q}{E_1^{3/2}}=q^1$, $\frac{D_c}{E_1}=D'_c$, $\frac{H_L}{E_1}=H_L'$ and $\frac{E'_2}{E_1}=E'_2$.

Now we have, $E_1=D_1+q^2/2gD_1^2$

$$\frac{E_1}{E_1}=1=\frac{D_1}{E_1}+\frac{q^2}{2gD_1^2E_1^3/E_1^2}=D'_1+\frac{q'^2}{2gD'^2_1}$$

whence $$q'=D'_1\sqrt{2g(1-D'_1)} \qquad \ldots.(9)$$

Similarly, we can derive the following further equations

$$D'_2=-\frac{D'_1}{2}+\sqrt{4D_1-\frac{15}{4}D'^2_1} \qquad \ldots.(10)$$

$$F_1^2=\lambda_1=\left\{\frac{D_c}{D_1}\right\}^3=2\left\{\frac{1-D'_1}{D'_1}\right\} \qquad \ldots.(11)$$

$$D_c'=D'_1\sqrt[3]{2\frac{1-D'_1}{D'_1}} \qquad \ldots.(12)$$

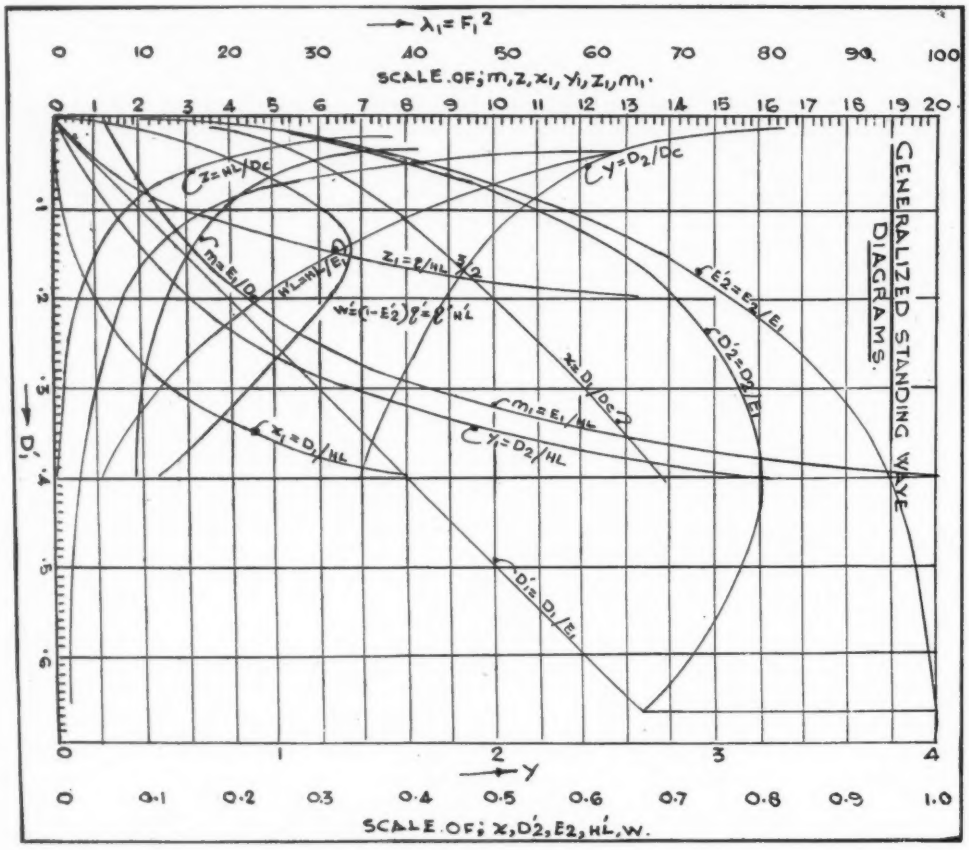
$$F_2^2=\lambda_2=\left\{\frac{D_c}{D_2}\right\}^3=\left\{\frac{D'_c}{D'_2}\right\} \qquad \ldots.(13)$$

$$H_L'=\frac{(D'_2-D'_1)^3}{4D'_1D'_2} \qquad \ldots.(14)$$

Appendix II

AUTHOR'S GENERALISED TABLE OF STANDING WAVE CO-EFFICIENTS.

D'_1	D'_2	q'	$\lambda_1=F_1^2$	D'c	$\lambda_2=F_2^2$	H'_L	E'_2	x	y	z	m	n	x_1	y_1	z_1	m_1	n_1	$q'H'_L$
.001	.0627	.008	1998	.0126	0081	.9366	.0634	.0793	4.98	74.4	79.3	5.03	0.0011	0.067	0.0089	1.067	0.067	0.0075
.002	.0880	.016	998	.020	.0116	.9088	.0912	.010	4.425	45.65	50.1	4.448	0.0022	0.097	0.0185	1.1003	0.1003	0.0146
.003	.1077	.024	665	.0262	.0148	.8850	.1150	.1145	4.120	33.95	38.1	4.45	0.0034	0.121	0.029	1.1212	0.1212	0.0213
.004	.1241	.032	497.5	.0317	.0167	.8740	.1260	.1260	3.92	27.66	31.63	3.96	0.0046	0.142	0.039	1.133	0.144	0.028
.005	.1390	.040	398.0	.0368	.0185	.8654	.1346	.1360	3.768	23.38	27.18	3.80	0.0058	0.162	0.0502	1.163	0.163	0.0346
.006	.1516	.048	331	.0415	.0205	.8520	.1480	.1447	3.644	20.78	24.10	3.68	0.0071	0.178	0.0608	1.172	0.172	0.0409
.007	.1633	.0559	285	.0460	.0223	.8398	.1602	.1522	3.550	18.200	21.784	3.584	0.0083	0.194	0.0725	1.191	0.191	0.0460
.008	.1739	.0639	248	.0503	.0243	.8245	.1755	.1590	3.468	16.400	19.90	3.500	0.0097	0.211	0.0851	1.214	0.214	0.0526
.009	.1844	.0719	220	.0542	.0255	.8165	.1835	.1659	3.39	14.96	18.41	3.43	0.011	0.226	0.0924	1.226	0.226	0.0587
.01	.194	.0798	196	.0583	.0272	.8030	.1970	.1720	3.325	13.70	17.07	3.37	0.0124	0.241	0.1110	1.245	0.245	0.0641
.02	.270	.1598	98	.0922	.0398	.7234	.2765	.2169	2.936	7.95	10.95	3.00	0.0276	0.372	0.2580	1.383	0.383	0.1157
.03	.327	.2371	64.67	.1204	.0466	.6669	.3331	.2490	2.710	5.54	8.32	2.78	0.045	0.491	0.4350	1.50	0.500	0.1581
.04	.372	.3141	48.0	.1454	.0597	.6148	.3852	.2750	2.580	4.23	6.87	2.64	0.065	0.602	0.6470	1.628	0.628	0.2100
.05	.412	.3900	38.0	.1680	.0678	.5750	.4250	.2972	2.450	3.42	5.95	2.53	0.087	0.717	0.8980	1.74	0.740	0.2242
.06	.446	.456	31.33	.1892	.0763	.5374	4626	.3174	2.360	2.845	5.285	2.445	0.112	0.830	1.1830	1.86	0.860	0.2510
.07	.476	.5417	26.57	.2089	.0844	.5020	.4980	.3351	2.280	2.400	4.78	2.38	0.139	0.943	1.5100	1.99	0.990	0.2720
.08	.504	.6158	23.00	.2275	.0920	.4726	.5274	.3516	2.210	2.08	4.40	2.32	0.169	1.064	1.8920	2.118	1.118	0.2915
.09	.529	.6890	20.22	.2452	.0996	.4443	.5557	.3672	2.160	1.815	4.07	2.25	0.202	1.185	2.315	2.25	1.25	0.3052
.10	.552	.7610	18.00	.2620	.1070	.4190	.5810	.3816	2.110	1.60	3.81	2.21	0.239	1.323	2.810	2.385	1.385	0.3190
.11	0.573	0.833	16.18	.2778	.114	.3970	.6030	.3960	2.064	1.430	3.60	2.17	0.277	1.442	3.615	2.52	1.52	0.3305
.12	0.593	0.9034	14.67	.2938	.122	.3718	.6282	.4085	2.02	1.267	3.41	2.143	0.324	1.600	4.01	2.69	1.69	0.3360
.13	0.611	0.9729	13.40	.3085	.1283	.3520	6480	.4214	1.9795	1.137	3.242	2.104	0.369	1.733	4.655	2.841	1.841	0.3421
.14	0.627	1.0419	12.29	.3231	.1370	.3230	.6710	.4333	1.945	1.02	3.095	2.075	0.426	1.905	5.51	3.04	2.04	0.3430
.15	0.643	1.110	11.32	.3365	.144	.3106	.6894	.4452	1.9090	0.922	2.968	2.046	0.483	2.07	6.43	3.22	2.22	0.3450
.16	0.657	1.1758	10.50	.3504	.152	.2920	.7080	.4566	1.880	0.835	2.86	2.025	0.550	2.25	7.44	3.42	2.42	0.3435
.17	0.671	1.2410	9.76	.3622	.159	.2759	.7241	.4690	1.851	0.761	2.759	1.998	0.617	2.384	8.54	3.625	2.625	0.3421
.18	0.683	1.3081	9.11	.3753	.167	.2568	.7412	.4788	1.815	0.690	2.665	1.975	0.696	2.65	9.97	3.86	2.86	0.3381
.19	.695	1.372	8.53	0.388	.174	.2442	.7558	.4895	1.7916	.624	2.576	1.952	.778	2.847	11.077	4.095	3.095	.336
.20	.706	1.4436	8.00	0.400	.182	.2300	.7700	0.5000	1.7654	.5741	2.500	1.9259	.874	3.08	13.10	4.35	3.35	.330
.21	.716	1.4977	7.52	0.4116	.188	.2160	.7840	0.5102	1.7402	.525	2.429	1.904	.972	3.316	14.401	4.63	3.63	.3235
.22	.725	1.5590	7.09	0.4220	.197	.2020	.7980	.5210	1.7190	.480	2.368	1.888	1.09	3.585	17.110	4.95	3.95	.3145
.23	.734	1.6200	6.70	0.4330	.206	.1900	.8100	.5300	1.6910	.438	2.310	1.872	1.211	3.860	19.560	5.26	4.26	.3080
.24	.742	1.6790	6.33	0.4440	.214	.1780	.8220	.5454	1.6724	.401	2.252	1.851	1.348	4.172	22.30	5.618	4.618	.2985
.25	.75	1.7300	6.00	0.4550	.223	.1670	.8330	.5503	1.6500	.367	2.200	1.833	1.496	4.494	25.39	5.98	4.98	.2890
.26	.756	1.7948	5.60	0.4633	.230	.1555	.8445	.5612	1.6460	.347	2.160	1.813	1.617	4.742	27.869	6.219	5.219	.2788
.27	.762	1.851	5.40	0.473	.240	.1455	.8545	.5712	1.604	.313	2.121	1.808	1.856	5.245	33.58	6.90	5.90	.2682
.28	.768	1.909	5.14	0.483	.249	.1362	.8638	.5800	1.591	.281	2.075	1.793	2.052	5.642	37.80	7.34	6.34	.2600
.29	.774	1.960	4.89	0.492	.258	.1260	.8740	.5890	1.572	.256	2.031	1.775	2.298	6.133	43.758	7.923	6.923	.2475
.30	.779	2.02	4.66	0.501	.265	.1160	.8840	.5987	1.560	.232	1.995	1.763	2.57	6.67	50.40	8.62	7.62	.2341
.31	.783	2.066	4.45	0.509	.275	.1093	.8907	.609	1.538	.215	1.965	1.75	2.836	7.163	57.392	9.149	8.149	.2261
.32	.786	2.118	4.25	0.518	.286	.101	.8990	.6173	1.517	.195	1.929	1.734	3.172	7.796	66.10	9.912	8.912	.2135
.33	.789	2.172	4.06	0.526	.297	.0052	.9048	.6270	1.499	.180	1.900	1.720	3.540	8.470	76.15	10.72	9.72	.2022
.34	.792	2.218	3.88	0.534	.307	.0860	.9140	.6360	1.482	.161	1.871	1.710	3.954	9.217	88.71	11.629	10.629	.1905
.35	.794	2.262	3.72	0.541	.317	.0792	.9208	.6460	1.467	.146	1.838	1.702	4.42	10.023	101.5	12.61	11.61	.1795
.36	.796	2.311	3.55	0.548	.326	.0730	.9208	.6460	1.467	.146	1.848	1.702	4.42	10.937	117.92	13.736	12.736	.1682
.37	.797	2.357	3.41	0.556	.340	.0670	.9330	.6651	1.434	.120	1.800	1.680	5.567	12.01	136.90	15.045	14.045	.1566
.38	.798	2.400	3.26	0.563	.351	.0604	.9396	.6750	1.421	.110	1.778	1.668	6.30	13.21	161.50	16.570	15.57	.1450
.39	.799	2.444	3.12	0.569	.362	.0580	.9420	.6838	1.405	.100	1.768	1.658	6.651	13.62	172.0	17.089	16.089	.1430
.40	0.80	2.490	3.00	0.577	.375	.050	.9500	.6934	1.386	.087	1.735	1.648	8.00	16.00	223.0	20.00	19.00	.1245
.45	.795	2.678	2.44	0.610	.451	.0290	.9710	.7379	1.306	.047	1.640	1.593	15.625	27.608	546.0	34.722	33.722	.0771
.50	0.781	2.837	2.00	0.630	.522	.0143	.9857	.7940	1.241	.023	1.586	1.563	35.00	54.67	1658.0	70.00	69.00	.0405
.60	0.724	3.042	1.33	0.660	.760	.0011	.9989	0.910	1.098	.0017	1.515	1.5133	545.00	658.0	83400.0	910.00	909.00	.0033
.667	.667	3.090	1.00	0.667	1.00	0	1	1	1	0	1.5	1.5	∞	∞	∞	∞	∞	0



$$E'_2 = 1 - H'_L \quad \ldots(15)$$

$$x = \frac{D_1}{D_c} = \frac{D'_1}{D'_c} \quad \ldots(16)$$

$$y = \frac{D_2}{D_c} = \frac{D'_2}{D'_c} \quad \ldots(17)$$

$$z = \frac{H_L}{D_c} = \frac{H'_L}{D'_c} \quad \ldots(18)$$

$$m = \frac{E_1}{D_c} = \frac{1}{D'_c} \quad \ldots(19)$$

$$n = \frac{E_2}{D_c} = \frac{E_1 - H_L}{D_c} = \frac{1 - H'_L}{D'_c} \quad \ldots(20)$$

$$x_1 = \frac{D_1}{H_L} = \frac{D'_1}{H'_L} \quad \ldots(21)$$

$$y_1 = \frac{D_2}{H_L} = \frac{D'_2}{H'_L} \quad \ldots(22)$$

$$z_1 = \frac{q}{H_L^{3/2}} = \frac{q'}{H'^{3/2}_L} = \frac{D'_1\sqrt{2g(1 - D'_1)}}{H'^{3/2}_L} \quad (23)$$

$$m_1 = \frac{E_1}{H_L} = \frac{1}{H'_L} \quad \ldots(24)$$

$$n_1 = \frac{E_2}{H_L} = \frac{1 - H'_L}{H'_L} \quad \ldots(25)$$

Thus giving different values to D'_1, all other dimensionless hydraulic jump co-efficients can be easily determined. These values are shown tabulated vide Appendix II for D'_1 from 0.001 to 0.01 at intervals of 0.001, from 0.1 to 0.4 at intervals of 0.01 and beyond that at intervals of 0.05. This table can be used directly for the solution of hydraulic jump problems in rectangular channels when any two of the elements are given.

Also the generalized table when represented graphically as in Figure 1 exhibits certain useful properties of the hydraulic jump. Thus from fig. 1 it can be seen that the highest value of $D'_2 = 0.8$ at $D'_1 = 0.4$ for which $\lambda_1 = F_1^2 = 3$, $H'_L = 0.05$, $z = \frac{H_L}{D_c} = 0.086$, $x = \frac{D_1}{D_c} = 0.6934$, $x_1 = \frac{D_1}{H_L} = 8$, $y_1 = \frac{D_2}{H_L} = 16$ and $z_1 = \frac{q}{H_L^{3/2}} = 223$. Bakhmeteff has classified the region with $\lambda_1 > 3$ and thus $D'_1 < 0.4$ as the zone of high kineticity and the region with

$\lambda_1 < 3$ and thus $\frac{2}{3} > D_1 > 0.4$ as the zone of the low kineticity. This distinction proves to be of great importance to a hydraulic engineer as it has been found experimentally by Bakhmeteff, Koch, Rehbock, Gibson and others that in the region of high kineticity, the jump occurs in the direct form, the experimental results are well in agreement with the theory and the loss of energy in the jump is fairly high. On the other hand in the region of low kineticity, the jump is of the undular form, the experimental results so far obtained are not so much in agreement with the theoretical results and the loss of energy is less than 5%, showing that the jump no longer serves the purpose of an effective energy destroyer and hence is hardly of any importance to the practical engineer. It is on this account that the dimensionless hydraulic jump co-efficients vide Appendix I and II respectively have been calculated by the author at closer range upto $x=0.7$ and $D'_1 = 0.4$ only. It is interesting to compare the extended Crump's co-efficient table given in part by the author vide Appendix I with the author's generalized table given vide Appendix II.

(*i*) The value of x ranges from 0 to 1, whereas the value of D'_1 ranges from 0 to $\frac{2}{3}$ only.

(*ii*) The jump is of practical importance to Engineer for values of x upto 0.7 and for D'_1 upto 0.4 only.

(*iii*) Table as given in Appendix II is more general in application than table vide Appendix I. Thus given any two elements of hydraulic jump, all other elements can easily be determined from table vide Appendix II, whereas for table vide Appendix I, one of the given elements must necessarily be the discharge intensity q so as to be able to obtain D_c.

(*iv*) The table vide Appendix II when represented graphically exhibits certain useful properties of the jump as already outlined whereas the table vide Appendix I does not.

(*v*) Table vide Appendix II is more convenient to use as compared with table vide Appendix I, when the given elements are q and H_L because $H_L^{3/2}$ can easily be obtained with the slide rule, or directly from table of halves power of numbers. On the other three hand the calculation of D, mathematically is more laborious and also from tables, because it would very often necessitate interpolation particularly when discharge intensity includes a fraction as well.

Illustration :—

Nangal Dam Data[5]—Determine D_2 for the following values of q and H_L.

q= 14.3 28.6 71.4 142.8 214.3 439.5 cusecs

H_L= 51.34 48.1 29.76 24.94 21.78 14.64 ft.

Now for q=14.3 and H_L=51.34, we have

$$z = \frac{H_L}{D_c} = \frac{51.34}{1.852} = 27.66 \text{ and } z_1 = \frac{q}{H_L^{3/2}} = \frac{14.3}{368.0} = 0.0389$$

Referring to tables vide Appendix I and II, we have,

when $Z=27.66$, $y = \frac{D_2}{D_c} = 3.92$, whence $D_2 = 7.27$ ft.

and when $Z_1 = 0.0389$, $y_1 = \frac{D_2}{H_L} = 0.142$, whence $D_2 = 7.29$ft.

Similarly the values of D_2 calculated for the remaining values of q and H_L are as follows :—

q cusecs.	14.3	28.6	71.4	142.8	214.3	439.5
H_L ft.	51.34	48.1	29.76	24.94	21.78	14.64
$Z = \frac{H_L}{D_c}$	27.66	15.7	5.50	2.92	1.97	0.81
D_2 ft.	7.27	10.2	14.85	20.80	24.68	33.8
$Z_1 = \frac{q}{H_L^{3/2}}$	0.0389	0.065	0.445	1.13	2.12	7.86
D_2 ft.	7.29	10.20	14.88	20.70	24.70	33.70

It will thus be seen that the values of D_2 calculated both from tables vide Appendix I and II are in very close agreement.

CONCLUSION

Extensive tables prepared by the author giving relation between the various dimensionless hydraulic jump co-efficients have been furnished in this paper. The merits and demerits of the extended table of Crumps dimensionless co-efficients given by the author vide Appendix I and the author's generalized table given vide Appendix II have also been discussed. The author hopes that the practical engineer would find these tables to be very useful for determining the elements of hydraulic jump particularly in the case of high spillway dams. The author's table vide Appendix II and the chart vide fig. 1 have the further advantage of being more general in application and of exhibiting certain useful properties of the jump.

ACKNOWLEDGMENT

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Considerations about the Calculations with the Total Energyhead in the Trapizoidal Profiles (with and without relative numbers)

This topic was suggested by Prof. Dr.-Ing. P. Franke, UNESCO-Expert, as a contribution to the Seminar of the Post-Graduate class of Hydraulic Engineering at the College of Engineering of Poona. The comparison of calculations with Energy head with the conventional method and that with the relative numbers of K. Flierl is given in this article. Editor.

By V. M. DESHPANDE

1. Introduction

1. (1) *Notation*

The following notation is used in this article

H = Total Energyhead.
Q = Discharge.
y = The depth of water in the channel.
v = Velocity of water.
k = Velocity head.
s = Bottom width of the channel.
m = side slopes of the channel.
F = Area of cross-section.
g = Acceleration due to gravity.
α = Correction factor for the velocity head.

1. (2) *General derivation*

For any cross-section the following relations hold good. The Energyhead

$$H = y + \alpha k$$
$$= y + \alpha \frac{v^2}{2g}$$
$$= y + \alpha \frac{Q^2}{2g \cdot F^2}$$

from which follows the well-known equation

$$Q = F \cdot \sqrt{\frac{2g}{\alpha}} \cdot \sqrt{H - y}. \qquad (1)$$

2. Critical Depth with the given Energyhead

The area of cross-section of the channel is

$$F = (s + my)\, y = sy + my^2$$

and from equation (1)

$$Q = F \cdot \sqrt{\frac{2g}{\alpha}} \cdot \sqrt{H - y}.$$

Hence the discharge

$$Q = (sy + my^2) \cdot \sqrt{\frac{2g}{\alpha}} \cdot \sqrt{H - y}. \qquad (1a)$$

For the critical depth, the discharge flowing through the channel must be maximum and hence for maxima the condition is

$$\frac{dQ}{dy} = 0.$$

Differentiating the equation (1a) w.r.t.y

$$\frac{dQ}{dy} = \sqrt{\frac{2g}{\alpha}} \left[(s + 2my)\sqrt{H - y} - \frac{(sy + my^2)}{2\sqrt{H - y}} \right] = 0$$

which gives

$$5my^2 + y(3s - 4mH) - 2sH = 0. \qquad (2)$$

The above equation gives the relation between the critical depth and the total Energyhead.

(1) For the rectangular cross-section m = 0, which gives $y_c = \frac{2}{3}H$, which is a well known result.
(2) For the triangular profiles s = 0, giving $y_c = {}^4/_5 H$, which is also already known.

3. Maximum discharge for given Energyhead

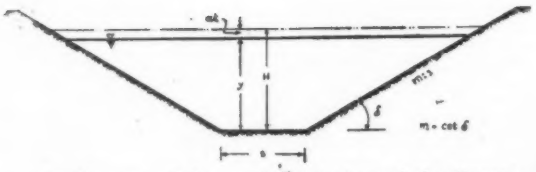
For the maximum discharge in a channel, the following relation is obtained—

$$Q_{max} = F_c \cdot v_c$$
$$= (sy_c + my_c^2) \cdot \sqrt{\frac{2g}{\alpha}} \sqrt{H - y_c}. \qquad (3)$$

For calculation of the maximum discharge, therefore, the critical depth must be known. The critical depth corresponding to the given Energyhead can be found out with equation (2). Thus knowing the critical depth and the energyhead, the maximum discharge can be calculated from the above equation (3).

4. Critical depth with the given discharge

For critical depth with given discharge the conditions satisfied are $H = H_{min}$ and $y = y_c$.



In the case of the trapizoidal channel the Energyhead is given as

$$H = y + \frac{\alpha}{2g} v^2$$

$$= y + \frac{\alpha}{2g} \frac{Q^2}{F^2}$$

$$= y + \frac{\alpha Q^2}{2g} \cdot \frac{1}{(sy + my^2)^2}$$

For the Energyhead to be minimum, the condition is

$$\frac{\partial H}{\partial y} = 0.$$

Therefore

$$\frac{\partial H}{\partial y} = 1 - \frac{\alpha Q^2}{g} \cdot \frac{(s + 2my)}{(sy + my^2)^3} = 0$$

and the final relation

$$\frac{(sy + my^2)^3}{(s + 2my)} = \frac{\alpha Q^2}{g} \qquad (4)$$

is arrived at.

(1) For the rectangular cross-section, $m=0$, which gives

$$s^2 y_c^3 = \frac{\alpha Q^2}{g}$$ which is the same as

$$y_c^3 = \frac{\alpha q^2}{g}$$ and is a well-known result.

(2) For triangular profiles, $s=0$, giving

$$m^2 y_c^5 = \frac{2\alpha Q^2}{g}$$

5. Minimum Energyhead for a given discharge

To calculate the minimum Energyhead corresponding to a given discharge, it is necessary to find out the critical depth for this discharge. This can be calculated with the help of equation (4). The minimum Energyhead can be computed from the following equation—

$$H_{min} = y_c + \frac{\alpha Q^2}{2g(sy_c + my_c^2)^2} \qquad (5)$$

6. Calculations of the two depths

The two water depths viz. one in the subcritical region and the other in the supercritical one are the roots of the equation

$$H = y + \frac{\alpha Q^2}{2gF^2} = y + \frac{\alpha Q^2}{2g(sy + my^2)^2}$$

which being of the fifth order has five roots. Hence it is advisable to calculate the two water depths with trial and error method as tabulated below. (Table No. 1)

7. Illustrations

It is proposed to solve two examples without the use of any relative numbers.

7. (1) *To calculate the two depths*

data :

Bottom width	$s=10.0$ m
Side slopes	$m=2$
Discharge	$Q=60.0$ m³/sec.
Energyhead	$H=3.0$ m
Correcting factor for velocity head	$\alpha=1$

To calculate the depths one has to solve the fifth degree equation as said previously. Hence the depths are calculated with trial and error method as indicated in the Table 1 below.

Table 1

1				2						3
$H=3.00$ m $=$ const.				$\frac{\alpha Q^2}{2g} = 0.051 \times 3600 = 183.6$						
4	5	$6=5^2$	7	$8=5+7$	$9=8^2$	$10=1/9$	$11=2\times10$	$12=4+11$	$13=2-12$	Remarks
y	sy	y^2	my^2	$F=(sy+my^2)$	F^2	$\frac{1}{F^2}$	$\frac{\alpha Q^2}{2g}=\frac{1}{F^2}$	$y+\frac{\alpha Q^2}{2gF^2}$	Diff.	
0.750	7.500	0.563	1.125	8.625	74.391	0.0135	2.471	3.221	−0.221	Depth in
0.800	8.000	0.640	1.280	9.280	86.118	0.0116	2.132	2.932	+0.068	shooting
0.794	7.940	0.630	1.260	9.204	84.640	0.0118	2.166	2.960	+0.040	region.
2.800	28.000	7.840	15.680	43.680	1908.0	0.000524	0.096	2.896	+0.104	Depth in
2.900	29.000	8.410	16.820	45.320	2099.4	0.000476	0.087	2.987	+0.013	tranquil
2.910	29.100	8.468	16.936	46.036	2118.9	0.000472	0.087	2.997	+0.003	region.

Table 1(a).

1			2			3			4
s=4,00 m			m=.1			$\frac{\alpha Q^2}{g} = \frac{400}{9,81}$ 40,76			
5	6=1×5	7	8=5²	9=2×8	10=6+9	11=10²	12=1+7	13=$\frac{11}{12}$	14=3−13 Remarks
y	sy	2my	y²	my²	F=(sy+my²)	F³	3+2my	$\frac{F^3}{(s+2my)}$	Diff.
1,22	3,88	2,44	1,49	1,49	6,37	125,00	6,00	20,83	+19,93
1,00	4.00	2,00	1,00	1,00	5,00	242,97	6,40	38,00	+ 2,76
1.22	4.88	2.44	1.49	1.49	6.37	258,00	6,44	40,30	+ 0,46

7. (2) *To calculate the critical depth with the given discharge*

Data :

Bottom width s= 4.0 m
Side slopes m= 1
Discharge Q=20.0 m³/sec.
Correcting factor for velocity head α= 1

For calculating the critical depth equation (4) has to be solved ; but this is also of the sixth degree and trial calculations are better. as in Table 1(a).

The critical area of cross-section from above table is 6.37 m². Hence the velocity is 3.14 m/sec. and the velocity head is 0.503 m. The minimum energyhead, therefore, is $H_{min.}$=1.723 1.72 m.

8. Calculations with the relative numbers of K. Flierl

8. (1) *General derivation*

The equation for the discharge in the trapizoidal channel can be written in the form as—

$$Q=(sH.\frac{y}{H}+mH^2.\frac{y^2}{H^2}).\sqrt{\frac{2g}{\alpha}}.\sqrt{H(1-\frac{y}{H})}.$$

$$=sH\,(\frac{y}{H}+\frac{mH}{s}.\frac{y^2}{H^2}).\sqrt{\frac{2g}{\alpha}}.\sqrt{H(1-\frac{y}{H})} \quad (6)$$

or alternatively as

$$Q=mH^2(\frac{s}{mH}.\frac{y}{H}+\frac{y^2}{H^2}).\sqrt{\frac{2g}{\alpha}}.\sqrt{H(1-\frac{y}{H})}. \quad (7)$$

It is evident that the terms $\frac{y}{H}$, $\frac{mH}{s}$ and $\frac{s}{mH}$ are non-dimensional factors.

8. (2) *Discussion of the relative numbers*

It can be seen that the factor $\frac{mH}{s}$ or $\frac{s}{mH}$ includes the slope—conditions m as well as the bottom width s and is constant for a particular given section. The factor $\frac{mH}{s}$ ranges between 0 and ∞ or the factor $\frac{s}{mH}$ ranges between ∞ and 0. The value of $\frac{mH}{s}$ can be conveniently chosen in the range of 0 to 1 with $\frac{mH}{s}=0$ for rectangular sections and $\frac{mH}{s}=1$ for trapizoidal section. The value of $\frac{s}{mH}$ in the range of 1 to 0 includes all other possibilities of variations of $\frac{mH}{s}$ i.e. in the range of 1 to ∞ ; with $\frac{s}{mH}=1=\frac{mH}{s}$ for trapizoidal section and $\frac{s}{mH}=0$ for triangular section.

9. Basic equations for the relative numbers

The relative numbers are denoted as—

$$\frac{y}{H}=f$$

$$\eta=\frac{mH}{s}$$

and $$\bar{\eta}=\frac{s}{mH}$$

Placing conveniently the given conditions in the range of 0 to 1 either with $\bar{\eta}$ or η, the following basic equations are obtained from equations (6) and (7)

$$\frac{Q}{sH.\sqrt{H}.\sqrt{\frac{2g}{\alpha}}}=f\sqrt{1-f}+\eta f^2\sqrt{1-f}$$

$$=6+\eta f^6*=\tau \quad (8)$$

and

$$\frac{Q}{mH^2 \cdot \sqrt{H} \cdot \sqrt{\frac{2g}{\alpha}}} = \bar{\eta}\bar{f}\sqrt{1-f+f^2}\sqrt{1-f} = \bar{\eta}\phi + f'\phi = \bar{\tau} \quad (9)$$

where f' and ϕ are the proportional factors by K. Flierl.*

10. Different slopes on two sides

If the slopes on two sides of the channel are different, e.g. m_1 and m_2, then follows—

$$F = by + \frac{m_1 + m_2}{2} y^2 = by + m_m y^2$$

consequently

$$\eta = \frac{m_m H}{s} \quad \text{and} \quad \bar{\eta} = \frac{s}{m_m H}$$

nad

$$\bar{\tau} = \frac{Q}{m_m H^2 \cdot \sqrt{H} \cdot \sqrt{\frac{2g}{\alpha}}} \quad (10)$$

Calculating the values of τ or $\bar{\tau}$ from the known conditions, f can be found from tables 2 or 3, and hence the depth as

$$y = fH \quad (11)$$

11. Critical depth with the given Energyhead

The equation for the critical depth can be written from equation (11) as—

$$y_c = f_c H$$

where the value f_c corresponds to the relative number τ_{max} or $\bar{\tau}_{max}$. For the condition $\eta = \frac{mH}{s} \leq 1$, from equation (8) follows

$$\tau = f\sqrt{1-f} + \eta f^2 \sqrt{1-f} = (f + \eta f^2)\sqrt{1-f}.$$

Differentiating for maximum and equating to zero

$$\frac{\partial \tau}{\partial f} = (1 + 2\eta f)\sqrt{1-f} - \frac{(f + \eta f^2)}{2\sqrt{1-f}} = 0$$

is arrived at, from which follows the condition

$$5\eta f^2 + f(3 - 4\eta) - 2 = 0 \quad (12)$$

For rectangular profiles $\eta = 0$ and hence $f_c = \frac{2}{3}$.

Similarly from equation (9)

$$\bar{\tau} = \bar{\eta}f\sqrt{1-f+f^2}\sqrt{1-f}$$

Differentiating and equating to zero the condition

$$5f^2 + f(3\bar{\eta} - 4) + 2\bar{\eta} = 0 \quad (13)$$

arrived at. For triangular section $\bar{\eta} = 0$. Substituting this in the above equation (13) $f_c = {}^4/_5$ is obtained.

12. Critical depth with the given discharge

For the calculation of critical depth with the given discharge the condition satisfied must be $Q = Q_{max}$ with $H = H_{min}$. The problem is to calculate the minimum Energyhead under which the given discharge is even possible and then the critical depth.

(a) For $\eta_c = \frac{mH_{min}}{s} \leq 1$ the value

$$\tau_{max} = \frac{Q}{s \cdot H_{min}\sqrt{H_{min}} \cdot \sqrt{\frac{2g}{\alpha}}} \text{ becomes with}$$

$$H_{min} = \frac{s}{m}\eta_c$$

$$\tau_{max} = \frac{Q}{s \cdot \left(\frac{s}{m}\right)^{3/2} \cdot \eta_c^{3/2} \sqrt{\frac{2g}{\alpha}}}$$

As η_c is an unknown value, taking it to the left-hand-side

$$\tau_{max}\eta_c^{3/2} = \frac{Q}{s \cdot \left\{\frac{s}{m}\right\}^{3/2} \sqrt{\frac{2g}{\alpha}}} \quad (14)$$

It can be readily seen that $\tau_{max}\eta_c^{3/2}$ is a non-dimensional factor, which depends on the shape and the discharge of the channel.

(b) Similarly for $\bar{\eta}_c \geq 1$ with the relation $\bar{\eta}_c = \frac{s}{mH_{min}}$ or $H_{min} = \frac{s}{m\bar{\eta}_c}$ the equation (9) can be put in the form

$$\frac{\bar{\tau}_{max}}{\bar{\eta}_c^{5/2}} = \frac{Q}{m \cdot \left\{\frac{s}{m}\right\}^{5/2} \cdot \sqrt{\frac{2g}{\alpha}}} \quad (15)$$

Comparing the right-hand-sides of the equations (14) and (15) it is evident that

$$\tau_{max}\eta_c^{3/2} = \frac{\bar{\tau}_{max}}{\bar{\eta}_c^{5/2}}$$

For triangular sections $s = 0$, which means that equation (14) has an infinite value. Hence with values of $\tau_{max}\eta_c^{3/2} > 1$, the reciprocal of it is to be calculated as

$$\frac{1}{\tau_{max}\eta_c^{3/2}} = \frac{\bar{\eta}_c^{5/2}}{\bar{\tau}_{max}} = \frac{s \cdot \left\{\frac{s}{m}\right\}^{3/2} \cdot \sqrt{\frac{2g}{\alpha}}}{Q}$$

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* A complete discussion of the relative numbers f and ϕ by K. Flierl is given in

1. Flierl, K.: Mitteilungen der Versuchsanstalt für Dasserbau der Technischen Hochschule München, Oldenbourg, München, 1953, No. 1, pp. 1-41.

2. Franke, P.: Calculation with the Energyhead, *Indian Journal of Power and River Valley Development*, Vol. VIII, No. 2 (Feb. 1958), pp. 19-24 and 26.

Knowing the values of $\tau_{max}\eta_c^{3/2}$ or $\frac{\tau_{max}}{\eta_c^{5/2}}$ from table 4 or 5 the value of η_c can be calculated easily and hence the minimum Energyhead from the expression

$$H_{min}=\eta_c \cdot \frac{s}{m} \text{ or } = \frac{s}{m.\eta_c} \qquad (16)$$

So also from the same tables the value of f'_c can be found and the critical depth

$$y_c=f'_c H_{min} \qquad (17)$$

can be computed.

13. Illustrations

The illustrations in the article 7 will be solved with the relative numbers, to show the ease with which the calculations can be done.

13. (1) *To calculate the two depths*

Data same as for 7.1.

Calculations :

$$\eta=\frac{mH}{s}=\frac{2,00\times 3,000}{10,00}=0,60$$

The relative number

$$=\frac{Q}{s.H\sqrt{H.\sqrt{\frac{2g}{\alpha}}}}=\frac{60,00}{10,00\times 3\sqrt{3}\times 4,429}$$

$= 0,2608.$

From table 2 the values of f' in tranquil and shooting regions are respectively

$f' = 0,9714$

and $f'' = 0,2650.$

Hence the depths in the respective regions are

$y' = f'H = 2,9142 = 2,91$ m

and $y'' = f''H = 0,7950 = 0,80$ m

13. (2) *To calculate the critical depth with the given discharge*

Data same as for 7. (2)

Calculations :

The non-dimensional factor

$$\tau_{max}\eta_c^{3/2}=\frac{Q}{s\left\{\frac{s}{m}\right\}^{3/2}\sqrt{\frac{2g}{\alpha}}}$$

$$=\frac{20,00}{4,00\times 4,00^{3/2}\times 4,429}$$

$= 0,1411.$

The values of relative numbers from table 4 are

$\eta_c = 0,430$ and $f'_c = 0,7117$

Therefore $H_{min}=\eta_c \cdot \frac{s}{m}=0,430\times 4,00=1,72$ m

and $y_c=f'_c H_{min}=0,7117\times 1,72=1,2241 \simeq 1,22$ m.

14. Conclusion

From the above discussion and illustrations, the advantages viz. the ease and quickness, of calculating with the relative numbers will be self-evident.

Table 2.

	$\tau=\sigma(1+\eta f')$					
f	$\tau=\sigma$ $\eta=0$	$\eta=0,2$	$\eta=0,4$	$\eta=0,6$	$\eta=0,8$	$\eta=1,0$
1,0000	0,0000	0,0000	0,0000	0,0000	0,0000	0,0000
0,9000	0,2846	0,3358	0,3871	0,4383	0,4895	0,5408
0,8000	0,3578	0,4150	0,4723	0,5295	0,5867	0,6440
0,7403	...	...	Maximum	...	...	0,6566
0,7326	...	...	,,	...	0,6009	...
0,7226	...	Maximum	...	0,5456	...	...
0,7095	...	,,	0,4909	...	...	...
0,7000	0,5834	0,4371	0,4908	0,5444	0,5981	0,6518
0,6917	...	0,4372	...	Maximum	...	...
0,6667	0,3849	...	...	,,	...	...
0,6000	0,3795	0,4250	0,4706	0,5161	0,5616	0,6072
0,5000	0,3536	0,3889	0,4243	0,4596	0,4950	0,5303
0,4000	0,3098	0,3346	0,3594	0,3842	0,4090	0,4338
0,3000	0,2510	0,2661	0,2811	0,2962	0,3112	0,3263
0,2000	0,1789	0,1860	0,1932	0,2004	0,2075	0,2147
0,1000	0,0949	0,0968	0,0987	0,1006	0,1025	0,1044
0,0000	0,0000	0,0000	0,0000	0,0000	0,0000	0,0000

Table 3.

$\bar{\tau}=\sigma(\bar{\eta}+f)$

f	$\bar{\tau}=\sigma f$ $\bar{\eta}=0$	$\bar{\eta}=0,2$	$\bar{\eta}=0,4$	$\bar{\eta}=0,6$	$\bar{\eta}=0,8$	$\bar{\eta}=1,0$
1,0000	0,0000	0,0000	0,0000	0,0000	0,0000	0,0000
0,9000	0,2561	0,3131	0,3700	0,4269	0,4838	0,5408
0,8000	0,2862 Max.	0,3578	0,4293	0,5009	0,5724	0,6440
0,7823	...	0,3586	...	Maximum	...	...
0,7683	...	...	0,4321	„	...	...
0,7570	...	Maximum	...	0,5064	...	...
0,7479	...	„	...	...	0,5813	...
0,7403	...	„	...	...	...	0,6566
0,7000	0,2684	0,3451	0,4218	0,4984	0(5751	0,6518
0,6000	0,2277	0,3036	0,3795	0,4554	0,5313	0,6072
0,5000	0,1768	0,2475	0,3182	0,3889	0,4596	0,5303
0,4000	0,1239	0,1859	0,2479	0,3098	0,3718	0,4338
0,3000	0,0753	0,1255	0,1757	0,2259	0,2761	0,3263
0,2000	0,0358	0,0716	0,1073	0,1431	0,1789	0,2147
0,1000	0,0095	0,0285	0,0474	0,0664	0,0854	0,1044
0,0000	0,0000	0,0000	0,0000	0,0000	0,0000	0,0000

Table 4.

η_o	$\Delta\eta_o$	f_o	Δf_o	$\tau_{max}\eta_o^{3/2}$	$\Delta(\tau_{max}\eta_o^{3/2})$
0,00		0,6667		0,0000	
	0,10		0,0135		0,0129
0,10		0,6802		0,0129	
			0,0114		0,0262
0,20		0,6916		0,0391	
			0,0097		0,0371
0,30		0,7013		0,0762	
			0,0082		0,0480
0,40		0,7095		0,1242	
			0,0070		0,0590
0,50		0,7156		0,1832	
			0,0061		0,0704
0,60		0,7226		0,2536	
			0,0053		0,0821
0,70		0,7279		0,3357	
			0,0046		0,0942
0,80		0,7325		0,4299	
			0,0042		0,1069
0,90		0,7367		0,5368	
			0,0036		0,1198
1,00		0,7403		0,6566	

Table 5.

$\bar{\eta}_o$	$\Delta\bar{\eta}_o$	f_o	Δf_o	$\frac{\bar{\eta}_o^{5/2}}{\tau_{max}}$	$\Delta\left(\frac{\bar{\eta}_o^{5/2}}{\tau_{max}}\right)$
1.00		0,7403		1,5231	
	0,10		0,0036		0,2814
0,90		0,7439		1,2417	
			0,0040		0,2569
0,80		0,7479		0,9848	
			0,0043		0,2309
0,70		0,7522		0,7539	
			0,0048		00,2032
0,60		0,7570		0,5507	
			0,0053		0,1739
0,50		0,7623		0,3768	
			0,0060		0,1426
0.40		0,7683		0,2342	
			0,0066		0,1095
0.30		0,7749		0,1247	
			0,0074		0,0748
0,20		0,7823		0,0499	
			0,0082		0,0401
0,10		0,7905		0,0098	
			0,0095		0,0098
0,00		0,8000		0,0000	

Note : For detailed 5 figure tables please refer Flierl, K. : Mitteilungen der Versuchsanstalt für Wasserbau der Technischen Hochschule München. Olden bourg, München, 1953, No. 1 pp. 1-41.

Commas instead of fullstops have been used to denote decimal point.

Hydraulic Turbines—II

DIMENSIONING

By N. VENKATA ROW

Impulse Turbines

The diameter D of the runner is taken as the diameter of the pitch circle—the points of contact of the jet(s). The peripherial speed of the buckets is usually 0.45 to 0.48 times the velocity of the jet. The velocity of the jet will be about 98.5% of the free spouting velocity

$$\frac{\pi DN}{12 \times 60} = \varphi\sqrt{2gH}$$

$$N = \frac{\varphi\sqrt{2gH} \times 12 \times 60}{\pi D}$$

$$= \frac{845\sqrt{H}}{D} \qquad (16)$$

Where D—pitch circle diameter inches
H—head feet
g—32.2
φ—0.45 to 0.48 (average 0.46)

The quantity (Q cusecs) discharged through a jet of d inches

$$Q = \frac{\pi d^2}{4 \times 144}\sqrt{2gH}$$

$$d = \frac{4.83\sqrt{Q}}{H^{\frac{1}{4}}}$$

Since $$Q = \frac{550\,HP}{62.5\,\eta H}$$

$$d = \frac{15.5\sqrt{HP}}{H^{\frac{3}{4}}}$$

with $\eta = 85.5\%$ (17)

Jets of 14 inches diameter have been used. With N_s and H being fixed (eq. 4) the speed may be increased by dividing the power between more jets. Since N_s is proportional to root of power the speed of the turbine is proportional to the root 4 root of number of jets.

$$N \alpha \sqrt{\text{Number of Jets}} \qquad (18)$$

For two jets the speed is increased by 41.4%, three jets 73.2%, four jets 100% and so on.

For horizontal arrangement for heads over 2000 ft. single jet is used and for lower two jets. For vertical arrangement two jets are used for heads over two thousand and may be more for lower heads.

$$Ns = \frac{845\sqrt{H}}{D} \cdot \frac{d H^{\frac{3}{4}}\sqrt{\eta}}{14.35\, H^{5/4}}$$

$$= \frac{54.5}{\frac{D}{d}} \qquad (19)$$

It will be seen that on the ratio (m) of the pitch circle diameter to the jet diameter the specific speed depends. The jet ratio can be selected as high as desired for slow running wheels. For fast running wheels values of 8 or 7 or even lower have been used. The highest degree of efficiency is obtained for values between 10 and 15 and decline taking place for lower values. A high value leads to a non-corroding runner but gives a lower speed. The highest ratio employed has been in the case of Glarus Plant in Switzerland. The turbine is rated for 3000 Hp to operate under a head of 4850 ft. at 500 RPM. The pitch circle diameter is 140 inches and the jet diameter is 1.5 inches.

Higher speed may be attained by increasing he number of jets but this results in a complicated lay-out and construction. The bucket dimensions have a direct relation on the jet diameter.

Width of the Bucket about	3.0 -3.5	d
Length of the bucket	0.8 -0.9	B
Depth of the bucket	0.27-0.3	B
Width of the cutout section	1.2	d

It will be seen from the data given above that with the increase of the number of jets the jet diameter decreases and the bucket dimensions become smaller. As the buckets form the most expensive part of the runner high speed means a reduction in the cost. On the other hand this saving is outweighed by more expensive construction due to the higher stresses in the revolving parts. It should also be borne in mind that the run-a-way speed increases, which has an influence on the unit design.

The number of buckets for the best efficiency :

$$Z = 0.5\frac{D}{d} + 15 \qquad (20)$$

Separate buckets form twin buckets, the centre portion of which halves the jet diverting it on either side. Impact on the back of the buckets which impairs the efficiency, is prevented as far as possible by carefully designed cutout sections at the inlet. The edges of these sections are highly stressed and are subjected to corrosion.

In the earlier designs the buckets were bolted to the central disc. Still such a design is being used by some manufacturers. Photoelastic studies of the

bolted bucket designs revealed some highly stressed parts at joints between the wheel disc and the bucket lug. After considerable experiments it was found that substantial reduction of stress at these critical points can be obtained by grooves located around the bolt holes of the wheel disc and the bucket lug.

Advances in the foundry technique have now made possible single piece runners with buckets cast integrally. Sometimes it is preferred to have the buckets cast in the form of a ring and then the ring fixed on to a central disc.

The shape of the bucket gives considerable difficulty in casting and all foundries will not be able to undertake the casting. To overcome this, steps were taken to design and manufacture welded plate buckets. Welded plate buckets were installed on the seven units of Big Creek Impulse Turbines. A recent further development has been the construction of integrally fabricated impulse runners to take place of integrally cast runners. In this type the bucket bowels are die-pressed of plate steel in one piece with a forged steel splitter welded in. The bucket bowels are then welded to two plate steel rings with notches cut into them so as to form lugs for supporting the bowels. This type of construction has been employed for the 39000 HP six jet San Bartolo Plant and the 140,000 Hp runner for the Kemano plant.

The casing for the runner must afford sufficient clearance between itself and the runner to prevent the water splashing back on the wheel. In some designs the upper part of the casing is provided with suitable insertions for diversion of the spray.

dam in the world, backsup the water in a lake. There are two power tunnels each 10 miles long 25 feet in diameter separated by 300 feet leading to a surge shaft. From here the water is led through penstocks each of 11 feet diameter and 4000 feet long. The wyes are of A.O. Smith design multilayer construction.

The power house is a vast excavated cave a quarter mile inside a mountain. The turbines are designed to operate at 327 RPM under a head of 2500 feet, each with four nozzles. For this particular head the units are the largest ever built. There were three different suppliers for the first three turbines. The constructional methods as well as the technical details differed from each other. The jet diameter was approximately $8\frac{1}{2}$" and the pitch circle diameter varied from 132 to $134\frac{5}{8}$".

	1	2	3
Construction of Buckets	welded	bolted	integrally Cast
Buckets number	24	22	23
Weight of the runner—tons	24	30	22
Average efficiency (90-150, 000HP) %	91	90.6	90.9
Maximum efficiency %	91.4	91.4	90.9
Maximum Output Hp	156000	157700	154300
Discharge cusecs Maximum	607.5	620.7	607

Data of some Installations

Station	*Country*	*Head ft.*	*Hp*	*RPM*	*m*	*Arrangement*
Dixence	Switzerland	5707	50000	500	33.9	H2RIN
Reisseck	Australia	5800	31000	750	20.1	—
Pallivasal	India	1877	6000	750	10.9	HIHIN
Pykara	do	2813	18750	600	13.1	HIRIN
Moyar	do	1218	18000	428	9.4	V2N
Kundah I	do	1040	28750	428	10.3	V5N
Kundah II	do	2340	50000	428	16.7	V3N
ElSalto	Canada	1350	13350	600	11.8	V4N
Bridge River	do	1115	62000	327	—	V6N

H2RIN stands for horizontal two runners each with one nozzle.

V2N stands for vertical two nozzle.

In the case of the Dixence a high value of m was chosen due to the limitations of the generator design.

Impulse Turbine Installations :

Kemano-Kitimat Hydro-Electric Project situated in British Columbia is a unique installation. This station is for an ultimate capacity of 2,240, 000HP—the largest under a private enterprise. This block of power is for the use of aluminium smelting —the largest smelter in the world.

The Kennedy Dam, the third largest rock filled

The generators were rated for 114 MVA. The protection system was very elaborate. The power generated was stepped up to 275KV for transmission over a distance of 70 miles.

An interesting feature of the turbines was that due to the nature of the load it was necessary to increase the load from zero to full value instantaneously. This requirement complicated the design as the hydraulics of the power tunnel and the pressure conduit systems necessitated the use of minimum turbine-nozzle-needle-opening time of 30 seconds. To meet the situation a special device was incorporated in the governor so that the needles may be opened

fully while the deflectors maintain the unit at synchronous speed even with no load on the generator. As soon as the load is thrown on the generator the deflectors automatically open at their maximum rate to permit the turbine to assume the full load. Loss of load will not lead to excessive rise in speed as tht deflectors will quickly divert the power jets from the buckets. This will permit the closure of the needles in relatively long time of about one minute, to suit the hydraulics of the penstocks and the tunnel.

The Chandoline Power House is the highest head impulse installation, second to Reisseck. For greater economic utilisation of the hydro potential Grand Dixence Societe Anonyme, Lausanne undertook extensive investigations for increasing the capacity of the Dixence lake. As a result they had planned extensive pumping stations and tunnels to feed the lake. The original lake is of capacity 50 million cubic meters and the dam of 80 meters height. With the extensions planned the dam will have to be raised to a height of 284 meters to store 400 million cubic meters of water. Instead of hightening the old dam a new one has been planned about 360 meters away from the old one. Such a plan will not cause any interruption in the running of the Chandoline station.

The Lake Grand Dixence will be feeding three generating stations Chandoline, Fionnay and the Nendaz.

The Chandoline station has five horizontal shaft impulse units. Each comprises of two impulse runners each wheel with a nozzle. Each unit develops 50,000 HP and designed for a maximum operating head of 1740 meters and a minimum head of 1560 meters. With the increase in the lake capacity the water level rises by 123.5 meters. As the units cannot work under such an increase of head a pressure reducer was planned. After extensive laboratory experiments with air as well as water as the working medium and also scaled model installation on Sernf pipe line Messrs Escher Wyss evolved two pressure reducers. These pressure reducers are for installation in a line branching off from the main tunnel leading to the other stations. Immediately after the pressure reducers (one a standby) is a surge shaft of 80 meters depth equal to the height of the old dam.

From the Grand Dixence lake is a tunnel of 5.3 miles length for a dischange capacity 59 cyds/sec. leading to another surge shaft. From the surge shaft penstock pipes take off to Fionnay station each pipe is 1360 yards in length and diameter ranging from 10 feet to 9 feet. The thickness at the start is 19/32" and nearer the station 2 1/8". There are six generating units each developing 77,200Hp at 428 RPM under a head of 2867 feet. Each unit comprises of two impulse wheels each wheel with a nozzle.

The water discharged from Fionnay is utilised for the Nendaz power station which houses six units each unit comprises of two impulse wheels. Each unit is of capacity 45000HP working under a head of 3306 feet.

Reaction Turbines:

Scroll case is the commonly used form of casing for the reaction turbines. Area of the casing is determined by the Vortex law i.e. the tangential component of the velocity multiplied by the radius at the particular point (measured from the centre line of the shaft) is a constant. The casing can be of RCC or cast iron or plate steel. The water velocity and smooth flow are the most important factors in the casing design as they affect the efficiency and the steadiness of operation. The water velocity depends on the head and the type of casing. For RCC the velocity is not to exceed 10 ft/sec and 14% of the spouting velocity. The corresponding values for cast iron and plate steeel casings are 20 ft/sec. 16% and 30 ft/sec and 23% respectively.

For Francis turbines the inlet diameter will be about 1.25 times the runner discharge diameter. The maximum overall diameter will be about 3.6 times the discharge diameter of the runner and the minimum value 3.1 times.

In the case of the Kaplan turbines, where large quantities of water are involved, the instake is to be designed with utmost care to minimise the losses. It may be necessary to install two piers to carry the rather heavy load of the structure.

INLET

Plant	*Width*	*height*	*area of the three openings*	*discharge cusecs*	*Velocity*
Kentucky	16'6"	39'	1902	9030	4.73
Fickwick	18'8"	41'	2292	11200	4.90
Wheeler	18'	41'	2052	9200	4.49
Chikamauga	17'8"	35'8"	1893	10700	5.65
Watts Bar	16'4"	37'8"	1848	8000	4.33

For Kaplan the ratio of the total width of the intake to the runner diameter will be about 3.

Speed ring is that part of the turbine which connects the cover plate and the discharge ring. In its simplest form it consists of upper and lower rings connected by stationary guide vanes or stay rods—hence also called the stay ring. This ring may either be cast separately or integrally with the casing or fabricated.

The guide vanes control the flow of water to the runner. Their number varies from 12 to 24 and always different from the number of buckets in the runner of the Francis, to avoid pulsations. Depending on the head these are made of cast iron or cast steel or bronze. There is tendency to fabricate the vanes also.

The stems of the vanes extend outside the cover and are linked by a shifting ring, which is turned by two connecting rods actuated by the governor.

Due to the governor action the vanes are in action continuously. If the water is not desanded silt will get between the vanes and the guide ring and erode both. When water carries considerable sand the guide wheel rings are lined with renewable facing rings.

To prevent interference between the runner and the wicket gates a space of about 3.5% of runner diameter is left in the case of Francis turbines.

As the specific speed decides the runner dimensions this has to decided first. For Francis turbines of high head type Morgan Smith formula (6) can be used whereas for other heads Moody's (5) gives fairly accurate results. With N_s fixed and consequently the speed, the next important figure is the discharge diameter. This helps in fixing the scroll case dimensions, draftube dimensions, spacing etc. For Francis:

From $$N = \frac{1840\,\phi\sqrt{H}}{D_1}$$

Where D1 is the diameter at the inlet in inches

ϕ coefficient varies from 0.65 to 0.9

For the Kaplan turbine the coefficient ϕ varies from 1.45 to 1.85 which means the peripherial velocity is greater than the free spouting velocity. This is due to the fact that the runner is placed in a contraction where the velocity is greater than the free spouting velocity.

Discharge = Area × velocity

$$Q = \frac{\pi D_2^2\,\phi_o\sqrt{2gH}}{4\times 144}$$

Where D2 is outlet diameter

ϕ_o velocity co-efficient at outlet

Incorporating the above in the basic specific speed formula and simplifying we get

$$Ns = 130\,\phi\,\frac{D_2}{D_1}\sqrt{\phi D\eta}$$

Runner outlet diameter

$$\frac{\pi D_2^2}{4\times 4\times 144}\times\phi_o\sqrt{2gH} = Q = \frac{550\,HP}{62.5\,\eta H}$$

$$D_2 = \frac{14.19}{H^{3/4}}\sqrt{\frac{HP}{\phi o\,\eta}}$$

ϕ_o varies from 0.20 to 0.33 for Francis and not greater than 0.7 for Kaplan

Designating h for the height of the gates, ϕ_g velocity coefficient of water passing through the gates

$$Q = \frac{\pi D_1\,h\,\phi_g\sqrt{2gH}}{12\times 12}$$

$$Ns = \frac{184\,o\phi\sqrt{H}}{D_1 H^{5/4}}\sqrt{\frac{\pi D_1\,h\,\phi_g\sqrt{2gH}\,\eta H}{12\times 12\times 8.8}}$$

$$= 260\,\phi\sqrt{\frac{h}{D_1}}\,\phi_g\,\eta$$

The co-efficient ϕ, ϕ_o, ϕ_g are variable depending on the runner design. The following emperical formula may be used for arriving at the ratio of the discharge diameter to the inlet diameter of the runner

$$\frac{D_2}{D_1} = 0.01\,N_s + 0.49 \qquad (21)$$

It will seen that the ratio varies with N_s. For high head Francis runners i.e., with low N_s, the inlet diameter will be higher to suit the speed without going to a peripheral velocity out-of-proportion to the water velocity. The quantity of water being small the height of the distributor will be small. For high speed Francis runners the inlet diameter will be smaller than the outlet diameter. The height of the distributor will be more, as a larger quantity of water has to be handled. The height of the wicket gates may be approximated to

$$N_s = 134\sqrt{\frac{h}{D_1}} - 17.5 \qquad (22)$$

With increase in specific speed the diameter at the inlet gets further reduced. The entrance and exit edges of the blades become continually flattened almost lie horizontal. Finally with the highest speed the band at the outer end of the vanes disappears. Narrower and fewer blades are used for reduction of friction. The runner then becomes the propeller type.

In the Francis turbines the flow is axial and in propeller is axial. A substantial reduction in friction looses has been possible in Propeller type by moving the inlet edges of the guide vanes into the whirl chamber. The water flows radially through the guide vanes and in the intermediate space it is deviated into axial direction.

Project.	*HP*	*Head ft*	*Ns*	*D1 cll in*	*D2 inches*	*h*	*N*
Boulder	114000	470	27.8	171	132	19	180
Dnieper	85800	113	70	182	230	78	88.2
Fontana	93300	333	32.6	—	144.5	29	150
Hiwassee	79500	189	48.3	161	165	—	120
Norris	69600	165	50.2	161	165	42	112.5
Wilson	36000	85	73.3	132	179.5	56	100
Bersimis	200000	875	26	144	102	14	277

The model ratio M of a given turbine is defined by the equation

$$M = \sqrt{HP}/H^{3/4} \quad (24)$$

The specific diameter Ds in inches or the discharge diameter of a homologous one is

$$D_s = 129/N_s^{0.37} \text{ For Francis} \quad (25)$$

$$D_s = 113/N_s^{0.34} \text{ For Propeller} \quad (26)$$

The required diameter is $D_2 = mD_s$ (27)

The distance between the centre line of the runner to the bottom of the runner (A) in the case of the Francis turbine and centre line of the runner to the centre line the distributor in the case of the propeller may be obtained from

$$\frac{A}{D_2} = \frac{N_s^{0.34}}{10.7} \text{ For Francis} \quad (28)$$

$$= 0.41 \text{ For Propeller} \quad (29)$$

$$= 0.37 \text{ For Kaplan} \quad (30)$$

In the case of the Kaplan turbines the discharge diameter may be expressed with a fair degree of accuracy by

$$D_2 = 55 \sqrt[3]{\frac{Q}{N}} \quad (31)$$

For the Francis turbine the constant is somewhat variable.

Example : Consider the case of a 23,000Hp turbine working under a head of 630 feet.

From Moody's formula the specific speed 26.6. The speed then will be 550 RPM, the next synchronous speed is 600.

$$M = \sqrt{\frac{23000}{630^{3/4}}} = 1.207$$

$$D_s = 37.15$$

$$D_2/D_1 = 0.756$$

Hence $D_2 = 45''$ and $D_1 = 59.5''$

The buckets or the blades in the case of the Francis turbine will vary from 21 in the case of low head installations to about 12 in the case of high head installations. The number of buckets (Z) :

$$Z = \frac{55}{\sqrt[3]{N_s}} \quad (32)$$

For the Kaplan turbines the hub details required to effect the blade adjustment offer many possibilities for variation and the problem of space and strength have been aggravated by higher heads and consequent increase in the number of blades. It is not the head along which fixes the number of blades. Blade length, shape and the specific loading and height of the centre line of the runner above the tail race are all factors which are to be taken into account. K.M.W. gives the guiding values.

Number of blades	*Head range in Ft.*
4	40-50
5	80-100
6	115-130
7-8	over 130

Different manufacturers adopt different number of blades depending on their design.

The exact value of the run-a-way speed can be predicted from Model tests in laboratory. R. E. Krueger of Bureau of Reclamation proposed

$$N_r = K.N.\left[\frac{H_{max}}{H}\right]^{\frac{1}{2}}$$

where $K = 0.003\ N_s + 1.7$

N_s = Specific speed

H_{max} = Maximum head

H = Working head

Unit spacing

The width of the unit block is determined in low or medium head installations by the hydraulic requirements and erection clearances. For high head installations the generator diameter decides the spacing.

Francis	Spacing from CL to CL	3.8 to 5 times the runner discharge diameter.
Propeller		3.3

Draft Tube

The draft tube connects the turbine with the tail race. As long as the difference in levels of the tail race and the turbine exit is within the atmospheric pressure the negative head in the draft tube adds to the positive head of the penstock to make-up the total head on the runner. Although the atmospheric head is 34 ft. the draft head is rarely kept over 15 ft. Draft tubes are never used in the cast of pelton wheels and hence the head from the nozzle to the tail race is lost.

The discharge diameter for constant head, power varies inversely as the specific speed, hence the greater the speed the greater is the velocity of water leaving the runner.

$$V = Q/\frac{\pi D_2^2}{4 \times 12 \times 12}$$

Where V is the velocity

Q discharge in cusecs

D_2 discharge diameter of the runner

Taking the case of the 60,000 Hp Bonneville Kaplan turbines having a discharge diameter of 280″ and discharging 12,000 cusecs.

$$12000 = V\frac{\pi D_2^2}{4 \times 12 \times 12}$$

$$V = 28.5$$

It will be seen that a substantial part of the energy still remains unutilised. The draft tube serves to decelerate the water with a minimum of loss so that the energy is regained. In the case of the above example of Bonneville turbines the water is decellerated to 5 ft. at the exit. For gradual deceleration to avoid eddies a minimum of length is necessary. With the increase in the quantity of water in the case of low head installations the shape of the draft tube deserves greater consideration. The head regained is

$$H_d = \frac{V^2 - V_s^2}{2g}\eta$$

where V velocity at the runner exit
V_s Velocity at the draft tube exit
n efficiency

The smaller is the value of V_s the higher the gain. For this reason the problem of expansion of the tube in the direction of the flow is of importance. For a given length excessive expansion results in the water stream separating from the tube sides. The intervening space will be filled with eddies. This will cause losses and consequently every care taken in the calculations will lessen the same.

The velocity ratio at the discharge end (ratio of the discharge velocity to the free spouting velocity) varies from 0.40 to 0.15 and at the draft tube exit the ratio varies from 0.10 to 0.02. Between a poorly designed draft tube and one of good design, the gain in efficiency will be as much as 2%.

The commonly used draft tube is of the elbow type. There will be a single pier or a double pier as the case may be.

A set of recommendations based on some of the existing installations :

Type	*Length of tube*	*Depth*	*Outlet width*	*Depth at outlet*
	In Terms of Discharge Diameter			
Francis	4.0	2.75	3.0	1.20
Propeller	3.85	2.70	2.50	1.15

Cavitation :

The setting of the turbine with respect to the tail race is of paramount importance as improper setting will lead to pitting which is caused by cavitation. The accompaniments of this are loss of efficiency of the unit, vibration etc. A case has been on record where the runners were so badly pitted by cavitation that they dropped off into the draft tube before it was realised what was happening.

Cavitation number or Thoma Number is defined by the equation

$$\sigma = \frac{Ha - Hm - Hs}{H}$$

H_a atmospheric pressure
H_m pressure at the runner discharge end
H_s static head (the distance between the tail race and the discharge end).
H head
cavitation co-efficient

Assume that both the head and the tail water levels are lowered by equal amounts. The head remains constant whereas the draft head changes so that H_m will progressively decrease to a point where it equals to the vapour pressure for that particular atmospheric conditions. The vapour under these conditions will fill the cavities. The formation of vapour pockets is called cavitation. Laboratory investigations on models have shown that the rate of form and collapse of the bubbles is of the order of 10,000 to 20,000 per second. The pressure produced by the collapse of the bubble is enormous which acting for a small duration on a small area causes destruction of the material.

Studies on experimental models with constant head and RPM and increasing static draft revealed that when a particular draft head is attained there is a fall in efficiency. This indicates a departure from normal conditions and the value for these conditions is the cavitation co-efficient.

For Francis turbines an empirical formula connecting the specific speed with the minimum cavitation has been proposed by Roger :

$$\sigma = 0.625 \left\{ \frac{Ns}{100} \right\}^2 \qquad (33)$$

In the case of the propeller turbine a runner of a given specific speed has an allowable plant sigma dependent on the blade area. A fair degree of reliance can be placed on the following formula :

$$\sigma = 0.28 + \frac{1}{7.5} \left\{ \frac{Ns}{100} \right\}^3 \qquad (34)$$

A value 10% higher than given from (34) will have to be taken for Kaplan. The above formulæ gives the lowest values that could be used. The actual values should exceed or at lest equal to the minimum.

There is yet another empirical formula :

$$\sigma_{min} = [3.6 \times 10^{-4} \times N_s^{4/3}] + [4.0 \times 10^{-6} \times N_s^{7/3}]$$

Thrust bearings

For supporting vertical water turbines thrust bearings are required which not only take the weights of the rotating parts—runner, shaft and the generator rotor—but also the hydraulic thrust.

The operation of the thrust bearing is based on the principle that an oil film is formed between two surfaces having a relative motion, due to the viscosity and adherence of oil to the surfaces. The first thrust bearing using that property appeared in the market in the names of Kingsbury and Mitchel. Both these contain tilting thrust pads, each mounted on a pivot as the stationery part of the bearing. The rotating part is a ring rigidly connected to the shaft by a collar and the usual key system.

The whole bearing is immersed in an oil bath. The rotating surface in contact with the oil in the gaps left between the pads washes the oil along with it in its rotation. The oil which is highly compressed gets heated up and consequently coolers are to be provided.

The hydraulic thrust may be calculated from :

$$Thy = CHD^2 \qquad (35)$$

Thy Thrust in lbs.
H Head in feet

[*Continued on page* 35

Assessment of Shearing Factors for Earthen Embankment Design

By S. N. AWASTHY

This paper deals with the derivation of methods for rapid determination of shearing factors i.e. angle of internal friction and cohesion of soils. The angle of internal friction of a soil is related with its constituents i.e. clay, silt and sand. In this connection for clay determination, a formula is obtained related to the plasticity index of the soil. The cohesion of a soil is also related with its plasticity index, and a formula is obtained.

The shearing resistance of the soil is the main asset for the stability of earthern slopes. In shearings tests, a straight line relationship is assumed to exist according to the Coulomb's equation ;

$$s = c + \sigma \tan \phi \qquad (1)$$

Where

s — shearing strength.
c—Unit cohesion.
σ—direct stress.
and ϕ—angle of internal friction.

In reality, this simplified assumption is only approximately true. The location of the straight line may vary widely with individuals.

Thus, the determination of 'c' and 'ϕ' of a remoulded soil compacted to the required degree as required in an earthen embankment, is not a simple affair.

Besides, the tests and condition of tests involved are delicate and take pretty long time.

The writer, therefore, proposes to assess the values of 'c' and 'ϕ' of the soils to be used in he construction by simpler methods without undergoing lengthy conventional procedure of shear tests.

ASSESSMENT OF 'ϕ'

The earliest classification of soils is based on particle sizes. Limits of various fractions of soil have been defined and each fraction named as sand, silt and clay under this scheme of classification. Opinions do not differ on these limits, the world over, as it will be exemplified by two representative opinions from Britain and U.S.A. quoted below :—

Table I.

CLASSIFICATION OF SOILS.[1]

M.I.T. Classification.	Clay.	Silt.	Sand.	Gravel.
B.S.I. Classification.	Clay.	Silt.	Sand.	Gravel.
Particle Size. (MM)		0.002 ←	0.06	2.0 →
B.S. Sieve No. (Retained on)			200 →	7 →

Thus, it is expected that engineerring soils will consist of a mixture of sand, silt and clay in various proportions. It can then be said that 'ϕ' of such a soil is a function of its constituents, viz. sand, silt and clay or stated mathematically :

$$\text{'}\phi\text{'} = f \text{ (sand, silt, clay)} \qquad (2)$$

If a soil containing sand only is considered, the above expression becomes $\phi = f$ (sand). Let the following relation be assumed :—

$\phi = x$. sand.

Where x is a constant.

Proceeding similarly with soils having only silt and only clay and assuming y and z as respective constants in these cases, the following expressions can be written :—

$$\phi = y. \text{ silt.} \qquad (4)$$

and $\phi = z$. clay. (5)

Equation (2) can now be written as :

$$\phi = x. \text{ sand} + y. \text{ silt} + z. \text{ clay.} \qquad (6)$$

Pit sand found in nature is adjudged to be the best. It is generally angular, well-graded ; undergoing selective deposition by sedimentation and is densified by long consolidation process in nature. Terzaghi and Peck[2] give a representative value of 'ϕ' of such sand as 46°. It is thus, easy to follow that inferior sand have value of 'ϕ' less than 46°. Therefore, the writer selects 46° as the upper limit of 'ϕ' for his work.

It can be visualised that the value of 'ϕ' also depends on the density of the soil. The maximum dry density is, of course, well-graded sand can be taken as 121 lb./cft.[3] In a generalised inference, which the writer is proposing to draw in this paper, a standard to which the work has to be accepted, the writer accepts 100 lb/cft. as a standard dry density.

Tschebotarioff[4] suggests that the angle of internal friction (ϕ) of the sand is directly dependent on the sand density and increase with it. This relation can be written symbolically as :—

$$\phi \propto \gamma_s$$

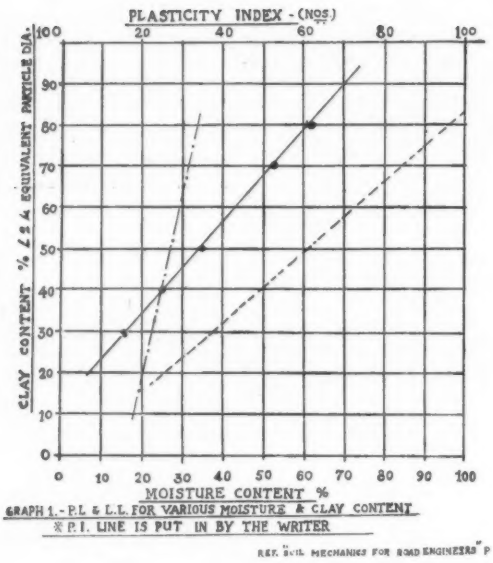
or $$\phi = k. \gamma_s \qquad (7)$$

where k is a constant. γ_s = max. density of sand = 121 lb/cft.

Substituting in equation (7) the values of 'ϕ' and 'γ_s' as accepted, the value of the constant, k, can be established as below :—

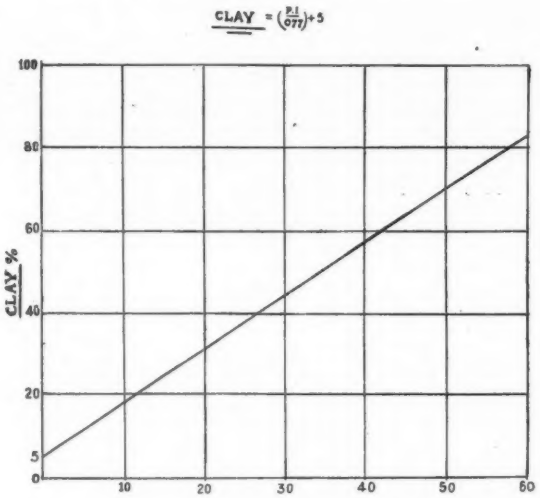
$$46° = k. 121$$

or $$k = 46/121 = 0.380 \qquad (8)$$



GRAPH 1.- P.L & L.L FOR VARIOUS MOISTURE & CLAY CONTENT
※ P.I. LINE IS PUT IN BY THE WRITER

REF. "SOIL MECHANICS FOR ROAD ENGINEERS" P 16



CLAY % – P.I. RELATION
GRAPH 2.

Now value of 'ϕ' for the accepted standard density of 100 lbs/cft. can be derived from equation (7) by putting the value of k as obtained in equation (8)

$$\phi = 0.38 \times 100 \qquad (9)$$

or $\phi = 38.0°$

In a soil containing 100 percent sand and having a density of 100 lb/cft, the coefficient, x in equation (3) will be:

$$38° = x.\ 100$$

or $$x = 0.38 \qquad (10)$$

Silt particles are physically and chemically rather similar to the particles in the sand fraction, the difference being due to the smaller size only. Like sand particles, their contribution to soil stability is is due mainly to internal friction, but slightly as well to cohesion which may develop owing to water films present between particles.

Saturated silt has low permeability and fails in the field usually under undrained conditions. Therefore, 'ϕ' values of silt should be taken as obtained by the consolidation quick shear test in which no time is allowed for draining.

Such value of 'ϕ' lies between 20° and 22°.[2] The writer adopts the lower figure of 20° for his work, to be on side of safety.

Silts with slight plasticity has maximum dry density greater than 100 lb/cft.[3] Since the writer has not come accross any more precise value than this, he adopts 100 lb/cft. as the upper limit of density for silt which incidently coincides with the standard density adopted for his work. Therefore, substituting the value of 'ϕ'=20° in the equation (4) for 100 percent silty soil, the value of coefficient y, is found as below:—

$$20° = y.\ 100$$

or $$y = 0.20 \qquad (11)$$

Experience has shown that natural masses of soft clay in the field, if sheared so rapidly that little drainage can occur, usually behave as if 'ϕ'=0°. At the present state of knowledge, it is recommended that the conservative value of $\phi = 0°$[5] be assumed But under long consolidation process in nature, however, clays may exhibit some frictional resistance. The writer therefore, feels that some value of 'ϕ', however small, will be developed in clay used in earthen embankments. He prefers to adopt small value of 'ϕ' not exceeding 5° as the upper limit for his work at the assumed standard dry density of 100 lb/cft.

Now the co-efficient z, for clay in equation (5) will be for 100 per cent clay as follows :—

$$5° = z.\ 100$$

or $$z = 0.05 \qquad (12)$$

To sum up, the equation (6) can now be given the following final shape:—

$$\phi = 0.38 \text{ sand} + 0.20 \text{ silt} + 0.05 \text{ clay} \qquad (13)$$

The results obtained from this formula are compared with the actual box shear test result. The difference is ± 10 per cent which is negligible for all practical purposes.[6] And therefore, the writer feels that his equation (13) can be safely used for finding out the values of 'ϕ' instead of performing the conventional shear tests.

The equation (13) for the assessment of 'ϕ' needs the percentage of sand, silt and clay in a soil sample. Sand percentage in a soil sample can be easily obtained by seiving. The silt and clay require wet analysis for their separation. But the wet analysis presents difficulties. The exact determination of clay seems to the writer to be almost a baffling problem, due mainly to difficult procedure and partly to inexperienced hand. This forced the writer to turn

to other easier means for finding out the clay fraction.

The liquid limit and the plastic limit and therefore, the plasticity Index (LL-PL) have certain relationship with the clay fraction of the soil, as may be seen from Graph (1) reproduced from "Soil Machanics for Road Engineers".[3] Generally it may be said that plasticity index is a function of the amount of clay present in a soil.

"When a soil contains a high proportion of organic matter, both the liquid limit (LL) and plastic limit (PL) are higher, although the plasticity index (PI) is similar to that of the same soil without an organic admixture".[5]

Thus, to relate clay fraction with LL or PL may be erroneous and it is safe to try to relate with PI. It may, therefore, be written as

PI is proportional to Clay.

In case PI of a soil is zero, it will necessarily mean that the clay fraction is also zero ; some percentage, however small, may be present there. Civil Aeronautics Administration (C.A.A.) classification[3] mentions that a free draining non-plastic sand can have 5 per cent of clay. (E.-1. group). Therefore, the writer prefers to allow 5 per cent clay in a soil to cause no plasticity for his work. Therefore, it can be written,

$$PI = k\ (Clay - 5) \qquad (14)$$

where k is a constant and is determined as shown in Table II with the help of the graph (1).

Table II.

DETERMINATION OF CONSTANT k FOR EQUATION No. (14).

Sl. No.	From Graph No. (1)			PI=(LL-PL)	$k = \frac{PI}{(Clay-5)}$
	Clay %	LL %	PL %		
1.	30.0	38.0	22.0	16.0	16/25=0.64
2.	35.0	44.0	24.0	20.0	20/30=0.67
3.	40.0	50.0	25.0	25.0	25/35=0.71
4.	45.0	56.0	26.0	30.0	30/40=0.75
5.	50.0	62.0	27.0	35.0	35/45=0.78
6.	55.0	67.0	28.0	39.0	39/50=0.78
7.	60.0	73.0	29.0	44.0	44/55=0.80
8.	65.0	79.0	30.0	49.0	49/60=0.82
9.	70.0	84.0	31.0	53.0	53/65=0.82
10.	75.0	90.0	32.0	58.0	58/70=0.83
11.	80.0	96.0	34.0	62.0	62/75=0.83

Average value of k=0.77 (Approx.)

Thus, the expression of clay determination takes the final shape as

$$PI = 0.77\ (Clay - 5)$$

$$\text{or } Clay = \left[\frac{PI}{0.77}\right] + 5 \qquad (15)$$

This equation is represented graphically in Graph No. (2) for quick work. It can be seen that if PI is zero, the clay content will still be 5 per cent as envisaged by the writer.

Sand is already known by seiving. Clay is determined from PI of a soil by equation No. (15), the remainder will be silt. In this way, all the three constituents of a soil needed for the determination of 'ψ' by equation No. (13) are known and hence 'ψ' can be easily computed.

ASSESSMENT OF "c".

Hogentogler[7] mentions that cohesion of a soil is indicative of its plasticity index. We can, therefore, say that c=f (PI). Let the following relation be assumed :—

$$c = k.\ PI. \qquad (16)$$

$$\text{or } k = c/PI \qquad (17)$$

where k is a constant.

values of 'c' for different types of soils tested by the writer are available from column 1 at densities shown in column 12 of Appendix I.[6] The values of "c" are recasted at the standard density of 100 lb/cft.[6] The value of k as found by the writer in his thesis is as 25.[6]

Substituting the value of k=25, in the equation No. (16) the final equation can be wrritten as :

$$c = 25.\ PI \qquad (18)$$

where c = standard value of cohesion at the standard density of 100 lb/cft.

Note : Comments on the paper are solicited, within 3 months after publication.

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Cross-Channel Power Link

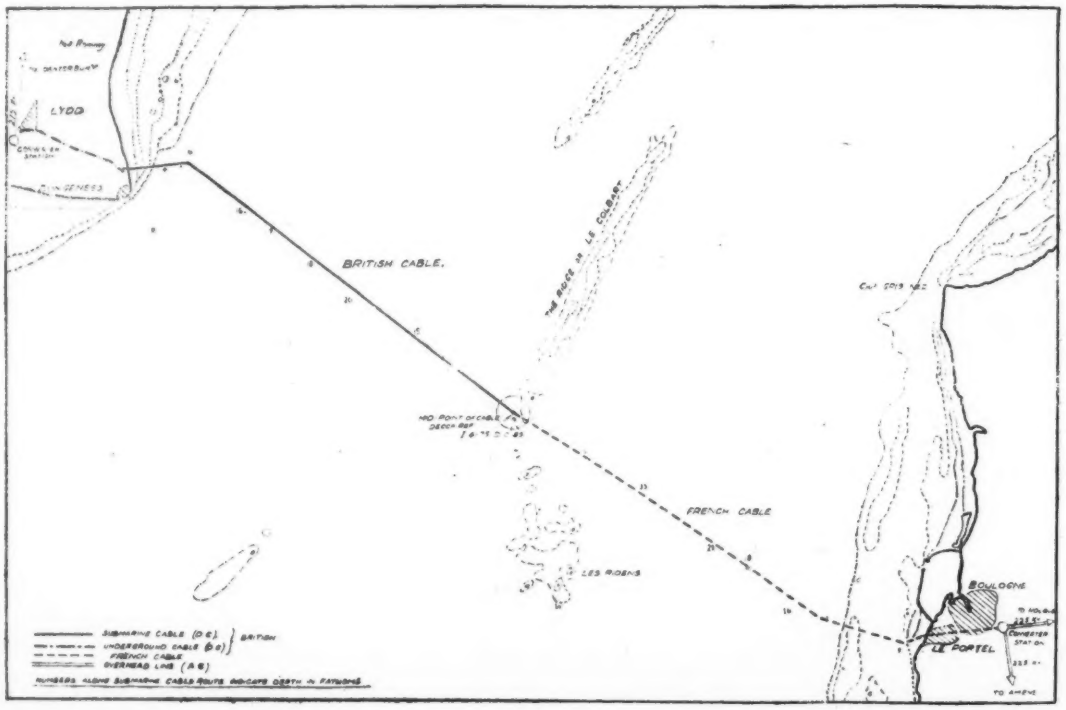
A £610,000 contract for cross-Channel power cables has been placed by the Central Electricity Generating Board with Associated Electrical Industries Limited and British Insulated Callender's Cables Limited. These two cablemaking concerns will lay half of the cross-Channel submarine power cable link which will connect the national electricity systems of Britain and France. The other half will be laid by cablemaking companies appointed by Electricite de France. Thus next year the egg with the lion on it may be boiled by melting Alpine snows, and the Paris Metro could at times be energized by coal from British collieries.

Differences in demand upon the British and French networks result in the availability of some hundreds of megawatts at peak winter loads. This is because of the marked dissimilarity in the patterns of electricity consumption here and in France due to differences of climate, clock times, hours of work and habits. As planned, the connection will transfer 160 megawatts of power from one system to the other at peak periods and transfer power at other times to meet conditions such as surplus or shortage of water at hydro stations. The ability of one country to call upon the other for extra generating capacity obviates the necessity to build expensive additional plant—to their mutual economic advantage.

The contract calls for the manufacture and laying of two similar 15-mile lengths of single-core submarine cable and two related land cables, the work being divided equally between BICC and AEI. The cables must be laid close enough to one another on the sea bed to conform with the Minister of Transport's requirement that at all times the magnetic compass deflection caused by the cables does not exceed two degrees at the sea surface.

To determine the practicability of laying the "go" and "return" cables simultaneously and closely spaced in a depth of some 20 fathoms and to confirm the practical technique for repairing the cables by the jointing-in of new lengths, sea laying trials were carried out in mid-Channel, about 10 miles off Dungeness, in the autumn of 1958. The Central Electricity Generating Board's 2,700-ton collier, m.v. Dame Caroline Haslett, was temporarily con-

Cross-Channel Interconnector Cable Route.



verted for these trials by the fitting of paying out and cable handling gear, and it is now proposed to use the same ship for the cable laying operations in the summer of 1961 and for repair operations. Additional alterations are required to equip the ship for this purpose and a contract for the work is being negotiated by the Generating Board with Associated Electrical Industries Limited.

Technical Details

The connection between the 275,000-volt Supergrid in this country and the 225,000-volt French network will be by a direct current circuit of 160,000 kilowatts, with a 20 per cent overlead for isolated periods of 30 minutes. The direct current transmission will operate at 200,000 volts between poles, with its centre point earthed, and the cables will be insulated for a nominal voltage of 100,000 volts to earth. The normal full load current will be 800 amperes. When complete, this submarine circuit will carry the heaviest d.c. load ever transmitted under water.

The Channel connection will consists of two 30-mile submarine cables routed between a point near Dungeness, Kent, and Le Portel, a suburb of Boulogne, with three-to four-mile sections of land cable at each end. Convertor stations able to rectify from alternating current to direct current and invert from direct current to alternating current are to be sited at Lydd, in Kent—where connection will be made to the British grid system through a 275,000-volt line to Canterbury—and at Echinghen near Boulogne. The d.c. circuit will go via underground cable from Lydd to a link-house at Dungeness. The circuit will also join the French 225,000-volt grid system by underground cable from the coast, at Le Portel, to the convertor station at Echinghen. The depth of the Channel over the British half of the route reaches a maximum of 20 fathoms, while the maximum in the French half is about 30 fathoms.

The cables themselves, as developed successfully from the sea trials, are of the impregnated paper insulated solid type. The submarine cable conductor is of 0.525 square inch in cross section and consists of stranded copper wire lapped with an electrostatic screen. The solid type impregnated paper insulation, also lapped externally with an electrostatic screen, is sheathed with lead alloy "E" which is protected against corrosion by vulcanized rubber tapes. Armouring is provided by a single layer of 0.232-inch galvanized steel wires with appropriate beddings and servings. The cables weigh about 39 lb per yard in air and about 28 lb per yard in sea water.

Anti-twist tapes will be applied over the lead sheath and under the anti-corrosion protection, these tapes being necessary to ensure that the cable is not damaged under the excessive tensions that may be required to lift the cable out of the sand on the sea bed during recovery after a possible fault. The design stress of these cables is about 150,000 volts/centimetre at ambient temperature.

The inferior heat dissipating properties of the soil compared with those of the sea necessitate using a larger conductor size for the land cable. Over sections of the route where the normal type of loam is found a conductor size of 0.6 square inch will be employed but, in the shingle areas which have very poor thermal dissipation properties, a 0.925 square-inch conductor will be necessary.

Land Connections

The land cable will be similar in design to the submariane cable, except that neither anti-twist tapes nor submarine anti-corrosion protection will be used, and the armour wires will be considerably smaller in size. The section of land cable from the link-house will, however, be joined to the submarine cable below high water mark for thermal reasons, and will therefore include anti-corrosion protection and have armour wires of the same size. Each country will be responsible for its own land section and half of the submarine section, from the coast to mid-Channel, where the cables will be joined together.

Thus the scene is finally set for one of the most ambitious submarine power transmission links ever to be installed. Work has already begun on the construction of the Lydd convertor station and by 1961, when the manufacture and laying of the submarine and land cables will have been completed, a new reserve of power will be obtainable economically in these two countries which are great industrial and domestic consumers of electricity.

It is estimated that the cost of manufacturing and installing the direct current cables and associated equipment will be about £4,600,000. Against this outlay there will be available to Britain at times of peak demand the equivalent of generating plant capacity which, if it had to be installed, would cost some £8,000,000. The cross-Channel interconnection is expected to result in net annual savings to this country of £250,000.

SOLIDAL

A new plastic insulated aluminium cable marketed under the name 'Solidal', has been developed by Northern Aluminium Company and its associates to make the best possible use of Alcan aluminium as a conductor and P. V. C. as an insulator. Its use for power supply and industrial installations above and below ground carries considerable advantages. It is not only an efficient conductor, but light, compact, inexpensive, and easy to install, for it can be laid and jointed with ease, and can be accurately set.

Solidal demonstrates almost all the characteristics that make aluminium of importance to the

electrical engineer: aluminium has a much higher current-carrying capacity, per pound weight and per pound sterling, than any other metal; it is durable requiring no painting; it is the cheapest non-magnetic metal; it is also easy to form and handle and readily welded, brazed and soldered. For further information about Solidal consult Alcan Asia Limited, the aluminium specialists, P.O. Box 2121, Hong Kong.

SOLIDAL

SPEED-RAMP

The first European application of a recently developed American step-saver, the moving sidewalk, has been placed in operation in a fashionable, ultramodern department store in Lucerne, Switzerland.

Capable of carrying more than 7,000 passengers per hour, the "Speed-ramp" connects a basement supermarket in the Innovation Department Store with the building's main entrance.

The passenger conveyor belt system was engineered and built by the Stephen-Adamson Manufacturing Company of Aurora, Illinois, U.S.A. and is equipped with a special passenger belt manufactured by the Goodyear Tire and Rubber Company.

Still relatively new in America, the passenger conveyor system has two major applications. Speedwalk systems developed jointly by the two firms, are in use where high pedestrian volume is to be moved over long distances on a horizontal plane. Speed-ramp systems, used singly or in multiples, are designed for similar high-volume locations where changes in level are desired.

Systems using both types of belt conveyors are in operation in Chicago, Los Angeles, New York City, Jersey City and in "Tomorrowland" at Disneyland. Others are being planned for shipping centers at Atlanta, Georgia; San Diego, California; and Aberdeen, Washington; where shoppers will ride to stores from distant corners of large parking areas.

The Speedramp installed in the Lucerne store was erected as a unit in the United States and shipped overseas to be installed.

RUBBER IN ASPHALT

A better national highway system at lower ultimate cost is possible through use of rubber additives in asphalt paving materials.

This was pointed out by research chemists of the Goodyear Tire and Rubber Company at a symposium on "rubber in asphalt". The meeting was conducted by the Asphalt Institute, a worldwide organization of more than 50 asphalt producing companies.

According to a paper prepared by Goodyear research men, the value of rubber in asphalt is proved by the more than 2,000 miles of rubberized asphalt roads now in use throughout the United States.

Roads built with rubberized asphalt have greater resistance to impact, shock and weather extremes. Such roads are capable of carrying the large volumes of high speed traffic contemplated for an expanded highway system of the future.

Actual road tests have proved the benefits of adding rubber to asphalt for both hot mix asphaltic concrete and seal coat construction.

Physical properties of asphalt pacing materials are greatly improved by rubber. This results in greater elasticity, ability to withstand fluctuating weather conditions and longer wearing road surfaces.

Another advantage is the ability of rubberized asphalt to stick to and hold many types of paving stone. Some of these cannot be held satisfactority by regular asphalt. This frequently makes it possible for road contractors to use locally available stone at significant savings.

In paving jobs all over the country, excellent road surfaces have been produced using low cost pea gravel with rubberized asphalt instead of much more expensive crushed limestone with regular asphalt.

Goodyear's experience with rubber in paving materials dates back to 1925, when the company surfaced a section of on of Ohio's busiest streets with rubber paving blocks. This pavement still is in excellent condition today.

Rubber paving blocks, however, proved too costly, but the success of America's first rubber pavement stimulated research along these lines. This led to development by Goodyear of Rubarite, a synthetic rubber mineral powder now commercially produced for paving applications.

Editorial

Engineers as Administrators

The time has come when engineers should review their position and capability regarding the execution of important projects. So far engineering has been mainly connected with the technical and scientific design and construction of projects and the connected functions of management, execution and administration have been assumed to fall in line with the requirements of the projects. Lately, however, the position has arisen where the executive and managerial hurdles have blocked the progress of an otherwise sound technical job. This sort of development is unfortunate because not only does it hamper progress, but it also shakes the faith of the people in engineering leadership. For this reason the perspective of engineers needs to be reoriented as discussed below.

So far the training of engineers has centred round the technical side, the theory and the formulæ needed and the technical assumptions made in the design of structures. This indeed is the essence of an engineer's training, but by itself it does not complete the objectives which have to be fulfilled in an engineer's career. There are a number of factors not connected with engineering technology but with economics and the human problems which are associated with the project. This being so, it is as important, if not more, for the engineer to be able to cope with the managerial aspect rather than the technical aspect only. But let us see how far an engineer's training incorporates this requisite. In the college right up to the master's degree, there is hardly a course in the syllabus bearing on economics or human relations. Even though the engineers rise to administrative positions at superintending engineer's level, they have not been put through an administrative course. This is a gap which needs to be filled up urgently.

There is so much frustration now about projects delayed, estimates exceeded and the potential not utilised, that we must secure a reorientation in the third Five Year Plan period. Firstly the curriculum of the universities should be revised to include for an engineering degree, economics, labour laws and public relations. Secondly the specific projects under execution should include chapters which deal with their effects and the economy of the region within overall economy of the country. Again the employment potential and the building of practical specialisation in the technical field must receive precedence.

Some private organisations—the oil companies for instance—have devised expensive courses for the training of their executives and management personnel. They insist that the officers of the rank of district managers, should be made to attend staff courses for which management experts are invited. In these courses, the officers are given certain recognised management principles set out in special literature so that they can apply this knowledge to the problems and operations which arise in their own sphere. Discussions are held in the manner of symposia or seminars and there is an informal exchange of ideas and opinions between the officers. The findings arrived at during discussions are further debated in the presence of the experts who participate in the proceedings simultaneously explaining the modern view points of management and the applications thereof to the specific problems posed. The same applies to the refresher courses which are convened by the Government of India agencies in respect of the Indian Administrative Service and the Indian Police Service. It is high time that similar courses were organised for engineers so that the projects are enriched by reviews which will be carried out by the engineers who uptil now have mainly contented themselves with giving the severely technical and design point of view to the projects.

There will be quite an extensive application of the principles of economics when these are brought to bear on the current problems of, say, the irrigation and power sector. To take one instance by way of illustration, it is being given out that due to various reasons, the revenues from the irrigation projects are dropping and such projects will in future be not so attractive financially. There is a fallacy in this view because it does not take into account the necessity for the upward revision of water rates which is now long overdue. It is well known that the yield from an irrigated acre is nearly one and a half times to twice the yield that can be had without irrigation. Similarly the rates of commodities have increased several times (upto six times) in the last 30 years. All the same there has been no proportionate increase in the water rates. The cost of construction and maintenance has gone up. Therefore, it is inevitable that the irrigation projects will be a losing concern till the rates are increased. Also there is power to be considered. Each multipurpose project gives irrigation, power and possibly flood control. There should be a balance of allocation of charges between all these subheads. If that is done the overall returns from a project involving control and regulated utilisation of water will be definitely remunerative. It is in this way that an engineer of today has to present his point of view and this goes much beyond the technical aspect of engineering.

For the refresher courses there will be an evident

necessity to obtain teachers of suitable experience and calibre. For this purpose we have to attach the highest priority to the best brains in the country being diverted to our universities and teaching institutions. The same has already been the practice in Germany, U.S.S.R., Britain and the United States of America. Similarly we should induce engineers who have reached an eminent position in their profession to devote their time to teaching. In this way they will be able to pass the torch of their knowledge to the succeeding generations, thus adding to the store house or know-how with which the problems of India's developing economy will be solved.

There is no doubt that for reaching high standards of quality we should obtain the services of foreign experts for some of our teaching institutions. But that should not reduce our own responsibility. The burden has to be ours first and last. Many of the retiring and retired engineers, who have got the credit of a life-time experience, should be induced to offer their thoughts and time to this national work. The Government and the other managerial leaders should make this offer attractive enough for the best talent to be brought into the forum of teachers in engineering universities and special advanced courses.

If the above policy is followed, there is hardly any doubt that a fair number of engineers will automatically find their way into the top most tiers of leadership in this country. In Russia, the United States and Britain, the position of minister is often occupied by engineers who have risen to the top ranks of their profession. The same should happen in India if we are to derive the fullest benefit of the resources which are limited. It is expected that with the focussing of attention on the achievement of physical targets, this aspect of the management of projects by engineers will come to the forefront and the fullest emphasis will be extended to the training of engineers in the field of administrative functions.

Hydraulic Turbines—II

[*Continued from page* 28

D runner discharge diameter in inches
C 0.26 for propeller
0.29 for Kaplan
0.03-0.11 for Francis

Another

$$Thy = \frac{CD^2 H_m}{2.94} \quad (37)$$

Hm maximum head
C Ns/250 for Francis
0.9 for propeller or Kaplan

The weight of the runner may be arrived at

$$\text{Wt. in Lbs.} = KD^3 \quad (38)$$

D diameter in inches
K 0-24 for single piece Francis
0.03 for large Francis units
0-009 for Propeller turbine
0.018 for Kaplan turbine

In the case of the Francis turbine the internal construction may be arranged so that the hydraulic thrust is minimised but for propeller and Kaplan turbines no reduction is possible. Since these axial flow runners consist of only the blades and a hub there are no balancing chambers or areas that limit the thrust. The design of the high capacity thrust bearings required for Kaplan turbines is significantly effected by the runaway speed.

Plant	*Bearing capacity in tons*
Kembs	700
Schwerstadt	900
Ottmarsheim	1300
Priest Rapids	1600
McNary	1740
Aswan	1600
Blondel	1400

With the development of Large Kaplan turbines the thrust bearings of increased capacity are being required. With bearings approaching 10 ft. diameter polishing methods of the surfaces have to be improved. Lapping process similar to that used in glass industry is now being adopted. Precision machinery installed in air-conditioned rooms optical straight edges make it now possible to manufacture the bearings to the highest degree of standards.

(To be concluded)

Our Contributors

Dr. B. K. R. Prasad has been our regular contributor and hardly needs any introduction.

V. M. Deshpande took the degree of B. E. (Civil) of the University of Poona in 1955 through the New Engineering College Sangli (now Walchand College of Engineering).

Served for three years in Bombay P.W.D. for practical experience. Worked in the College of Engineering, Poona, where he studied M.Sc. (Eng.) under the UNESCO-Expert Dr. Ing. P. Franke. At present Lecturer in Civil Engineering, Government Polytechnic, Amravati. His last contribution in our journal was in October, 1959.

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Mr. N. Venkata Row's first article in the series was published in our March, 1960 issue. His concluding article will be published in the August issue. Mr. Row is with M/s Larsen and Toubro Limited, posted at their Delhi office.

Prof. R. M. Advani graduated from the University of Bombay in 1944. After two years' experience in Bombay P.W.D., he joined the staff at N.E.D. Engineering College, Karachi, where he continued till 1947. From 1948 to 1954, he was Lecturer in Civil Engineering at the University of Roorkee from where he came over to Jodhpur as Professor of Civil Engineering at the M.B.M. Engineering College, Jodhpur. He is engaged in Hydraulic work and has contributed over a dozen papers in various scientific and technical journals. He contributed to our journal in August, 1958.

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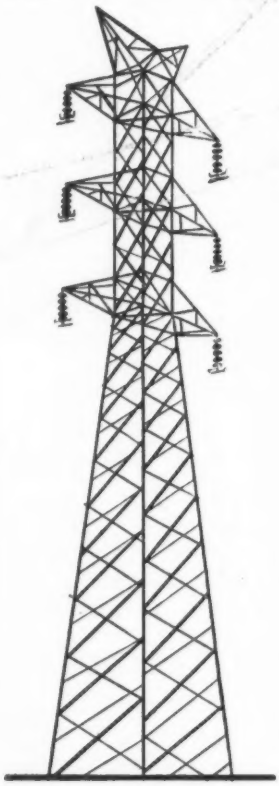

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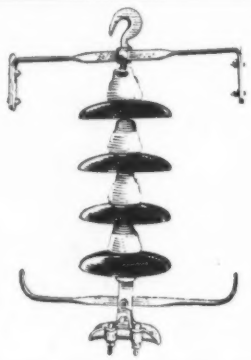

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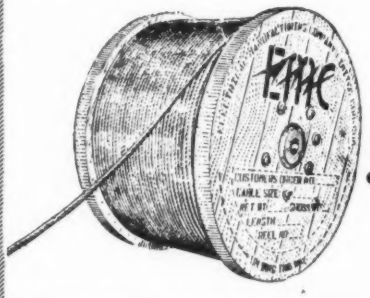

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